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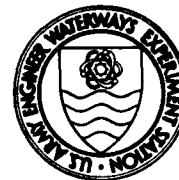
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**CONCRETE AND ROCK TESTS
MAJOR REHABILITATION, DRESDEN ISLAND
LOCK AND DAM, ILLINOIS WATERWAY
CHICAGO DISTRICT, PHASE I REHABILITATION**

by

R. L. Stowe, G. S. Wong, B. A. Pavlov

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U. S. Army Engineer Waterways Experiment Station
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September 1980

Final Report

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ROCK TESTS, DRESDEN ISLAND LOCK & DAM, PHASE I REHABILITATION

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20. ABSTRACT (continued).

weak zones in the foundation are clay seams in the broken limestone and a shaley clay layer underlying the limestone.

New concrete is present at a number of locations as patches or overlays; it is in good condition. Old concrete is lightly to severely deteriorated. About 80 percent of the exposed vertical surfaces in the lock and dam has been affected by frost action to varying degrees. The average depth of concrete deterioration in the lock walls is 0.7 ft; in the upper gate bays, 1.5 ft; in the arch dam future lock walls, 1.3 ft; in the spillway dam abutment, 1.0 ft; in the ice chute pier, 2.0 ft; in the head gate piers and sill, 1.0 ft; and in the upstream one-half of the tainter gate piers, 2.3 ft. Maximum depth of damaged concrete is 3.1 ft. The damaged concrete primarily resulted from cycles of freezing and thawing. The concrete beyond the damaged zones is structurally sound.

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PREFACE

This testing program, "Concrete and Rock Core Tests, Major Rehabilitation of Dresden Island Lock and Dam, Illinois Waterway, Chicago District, Phase I, Rehabilitation," was conducted for the U. S. Army Engineer District, Chicago. The work was authorized by DA Form 2544 No. NCC-IA-77-32, dated 5 April 1977.

Drilling was conducted by personnel of the Soils and Pavements Laboratory (S&PL) of the U. S. Army Engineer Waterways Experiment Station (WES) during the period April 1977-May 1977 under the direction of Mr. Mark Vispi. Laboratory tests were performed at the Concrete Laboratory (CL) and the S&PL during the period June 1977-August 1977 under the direction of Messrs. Bryant Mather, Chief of CL, and John M. Scanlon, Chief, Engineering Mechanics Division. Mr. G. P. Hale supervised the laboratory testing conducted in the S&PL; and Mr. G. S. Wong conducted the petrographic examination. Mr. R. L. Stowe was Project Leader and was assisted in performing laboratory work at the CL by Messrs. F. S. Stewart and J. B. Eskridge, and Ms. B. A. Pavlov. This report was prepared by Messrs. Stowe and Wong and Ms. Pavlov.

The Commanders and Directors of WES during the conduct of the investigation and the preparation and publication of this report were COL J. L. Cannon, CE, and COL Nelson P. Conover, CE. Mr. F. R. Brown was Technical Director.

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APPENDIX A: ABBREVIATIONS

EXHIBIT A: PHOTOGRAPHS OF CORES

EXHIBIT B: FIELD DRILLING LOGS

NOTE: Exhibits A and B are on file at the Chicago District office.

CONVERSION FACTORS, INCH-POUND TO METRIC (SI)
UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.02831685	cubic metres
degrees (angle)	0.0174533	radians
feet	0.3048	metres
feet per second	0.3048	metres per second
miles (U. S. statute)	1.609344	kilometres
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (force) per square foot	0.09511274	megapascals

CONCRETE AND ROCK CORE TESTS, MAJOR REHABILITATION OF
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY, CHICAGO DISTRICT
PHASE I, REHABILITATION

PART I: INTRODUCTION

Location of Study Area

1. The Dresden Island Lock and Dam is located in Grundy County, Illinois at mile 271.5 on the Illinois River, about 2-1/2 miles downstream of the junction of the Kankakee and the Des Plaines Rivers. The city of Joliet, Illinois is some 14 miles upstream from the dam. Driving distance from Chicago is about 50 miles.

Background

2. At a meeting held at the offices of the Chicago District Corps of Engineers (NCC) on 11 February 1977, representatives of the Concrete Laboratory (CL) and the Soils and Pavements Laboratory (S&PL) of the Waterways Experiment Station (WES) were requested to submit a proposal for work to assist the Chicago District in connection with concrete and rock exploration and laboratory testing. The work would be accomplished in two phases. Phase I work concerned concrete and rock exploration, laboratory testing for a major rehabilitation of the Dresden Island Lock and Dam. Phase II work concerned drilling and laboratory testing to comply with certain Office, Chief of Engineers (OCE) and North Central Division (NCD) comments as outlined in Reference 1. The name and affiliations of the attendees at the 11 February 1977 meetings are shown in the following tabulation. This report presents the results of the Phase I work.

Fred Paterson	NCD
Terrence Smith	NCD
Ignas Juzenas	CDO
Jim Przewoznik	CDO
Richard Huelsman	CDO
George Sanborn	CDO
C. Ruiter	CDO
Richard Stowe	WES
Mark A. Vispi	WES

NCD - North Central Division
 CDO - Chicago District Office
 WES - Waterways Experiment Station

3. It was explained that the major work effort during the rehabilitation phase would be resurfacing most of the exposed concrete surfaces of the lock and dam. To assist in this effort, WES was requested to determine the extent of deteriorated concrete on these surfaces. The structures and number of borings to be drilled were assigned by the Structural Section of the District; drilling assignments were made for both the concrete and rock cores. The specific boring locations on the separate structures were assigned by the author. These assignments were made after a field inspection, review of an inspection report¹, and a reconnaissance report published in January 1977.² A secondary effort was to obtain engineering design parameters, shear strength envelopes, and modulus of elasticity values of the foundation rock under certain structures. These data supplement previously reported data presented in References 3 and 4.

4. The major work effort during Phase II will be to conduct supplemental tests of foundation rock for purposes of developing strength envelopes based on direct shear tests. The results of these tests will be used by the District to check previous structural stability analyses.

5. In the summer of 1976, concrete and shotcrete cores were taken from the lock chamber walls at Dresden Island Lock; the cores were tested

at the WES. The cores were examined for extent of deterioration and tested for strength and modulus of elasticity. Some of the results of the examination and tests are included in this report for completeness. The complete report can be found in Reference 2. No cores were taken inside the lock and chamber during this investigation. As a result of a field inspection in July 1977, conducted by the Chief, Structural Section, and the author, eight additional borings were selected. The results of tests on the core taken from these borings will be included in a supplement to this report.

Objectives

6. The objectives of this study were to:
 - a. Conduct drillings for laboratory testing of concrete and foundation rock.
 - b. Make an analysis of test conducted, a summary of the concrete condition over the project site, and a summary of foundation at certain locations.

Scope

7. The drilling was accomplished using a WES drilling crew, plant, and supplies. Chicago District Con-Ops supplied the floating plant assistance. A Bureau of Reclamation geologist on contract to WES logged and preserved the core for testing. Delivery of the core to WES was made in two shipments, one in mid May 1977 and one in mid June 1977.

8. The objectives of this study were accomplished by drilling concrete and rock core and sampling backfill material, conducting petrographic examinations, characterization property tests (unconfined compression, splitting tensiles, unit weight and ultrasonic pulse velocity), and engineering design property tests (modulus of elasticity, Poisson's ratio and direct shear). Direct shear tests were run on concrete to rock core, intact core, precut surfaces of rock and on specimens oriented to obtain cross-bedded shear strength. One clay sample was subjected to a petrographic examination for purposes of determining mineral content.

PART II: DRILLING AND EXPLORATION

Previous Exploration

9. The core drilling by WES in 1976 is the only sampling of concrete from which the samples are currently available that are pertinent to this study. Sixteen horizontal borings were drilled into the lock chamber walls from inside the chamber. The approximate boring locations within monoliths are shown in Plate 1. The monolith numbering system is the same as used in Plate 9, Reference 3.

10. Previous foundation explorations were completed in 1971⁴ for purposes of: (a) obtaining a foundation appraisal of the bedrock and backfill of the lock and dam and (b) provide design parameters for use in a structural stability analysis. Four borings were put down, two in the lock structure and two just upstream of the dam structure. The geologic information obtained from these borings is presented in Reference 4 and serves as a valuable reference for the work accomplished during this investigation.

Current Drilling

11. Drilling operations were conducted at two different times. The first operation began on 13 April 1977 and ended on 3 June 1977 with a total of 48 borings being made. The second operation began 4 June 1977 and ended 8 August 1977. Eight additional short borings were drilled in the tainter-gate piers. Test results from specimens recovered from these borings are included in this report. As mentioned earlier 16 borings were drilled in the summer of 1976 into the lock chamber walls from inside the chamber; 2 of the 16 were in the shotcrete section of the river wall. The shotcrete core showed an excellent bond to the old concrete and is considered extremely well placed. The job at Dresden Island Lock is considered the best example of shotcrete application within the Corps.

12. The general boring location plan is presented in Figure 1a and 1b; the specific locations on structure sections are given in Plates 1, 2a, and 2b. A summary of boring locations, direction, number and depth is given below:

	<u>Location</u>	<u>Direction</u>	<u>No. of Holes/ Depth, ft</u>
Lock Structure	Lock chamber land wall	H*	7/3
	Lock chamber river wall	H	9/3
	Sloping river wall face	H	2/5
	Vertical river wall face	H	3/5
	Miter gate bays	H	2/3
Dam Structure	Arch dam abutment	H	4/3
	Spillway dam	H	1/3
	Tainter gate piers	H*	13/3
	Tainter gate piers	V	2/11
	Ice chute pier	V	1/3
	Head gate pier	H	4/3
	Head gate sills	V	2/5
Upper & Lower Approach Walls	Upper	H	1/3
	Lower	H	5/5
	Lower	V	2/48
	Lower	V	1/33
	Lower	V	2/3
	Lower, in backfill	V	3/25

* Horizontal = H;
Vertical = V

13. A total of 23, 27, and 14 holes were put into the lock structure, dam structure, and upper and lower approach walls, respectively; these numbers include the borings made in the summer 1976.

14. Drilling equipment consisted of an Acker Toredon Mark II skid-mounted rotary drill rig, and a Concord portable drill rig. Six-inch inside diameter diamond core bits and a 5-ft long double tube core barrel was used to drill the concrete and the bedrock in the deep holes. A single core barrel was used with a Concord rig to drill the shallow holes. Access to the drill holes was by a marine floating plant and for holes on top of structure by crane. Typical drill rig setups are shown in Figure 2.

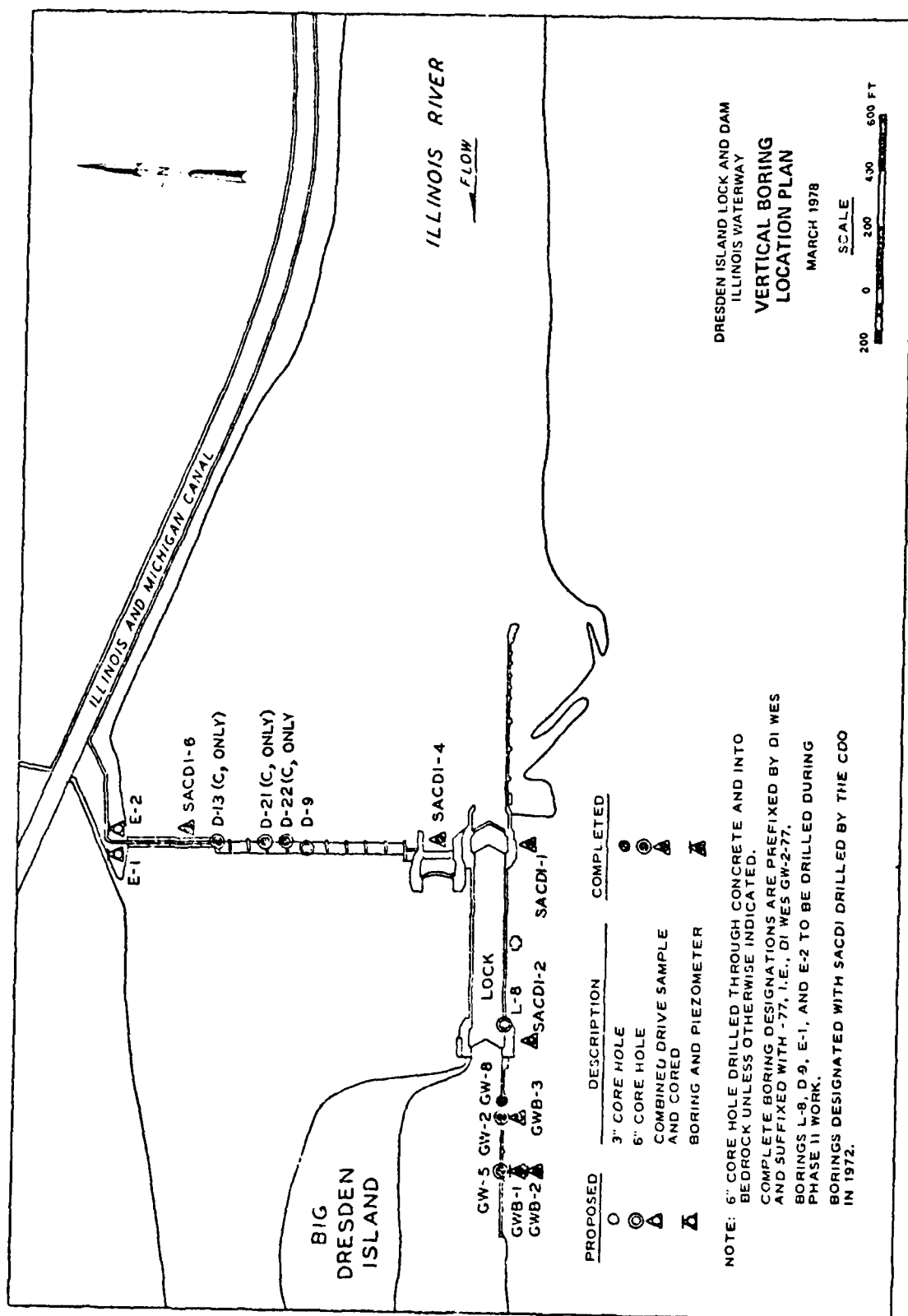


Figure 1a

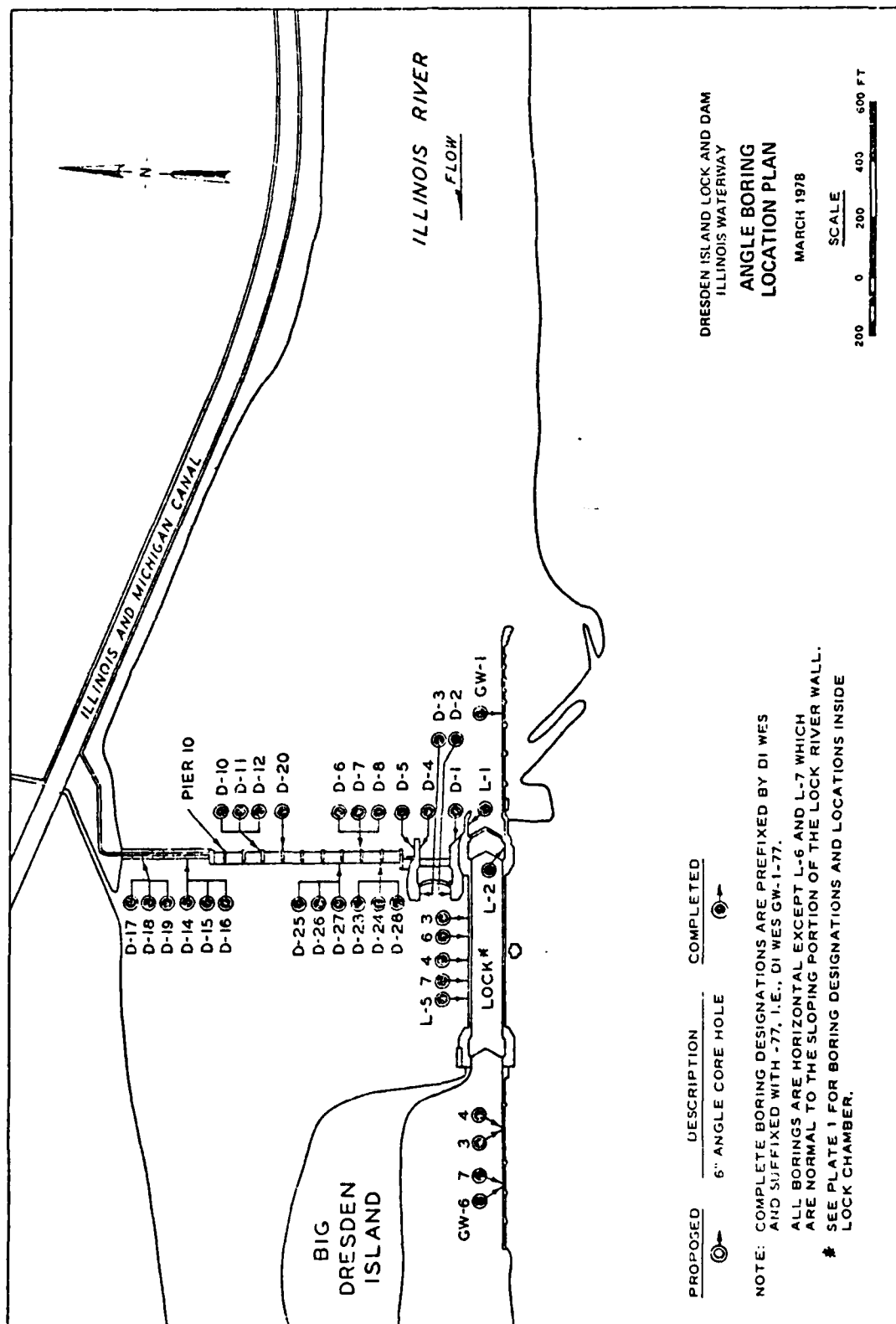
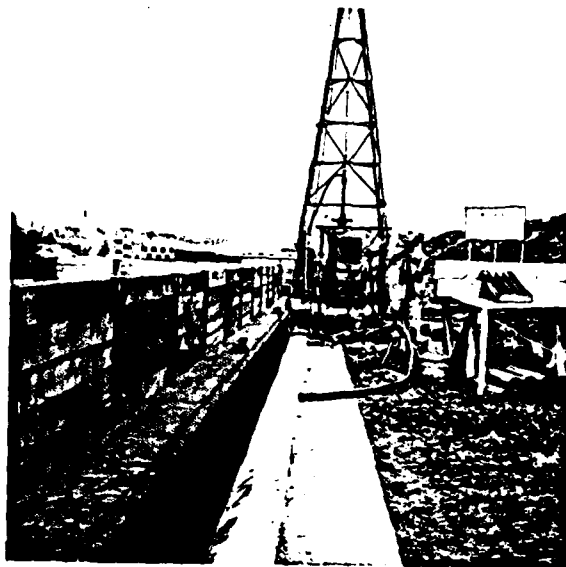
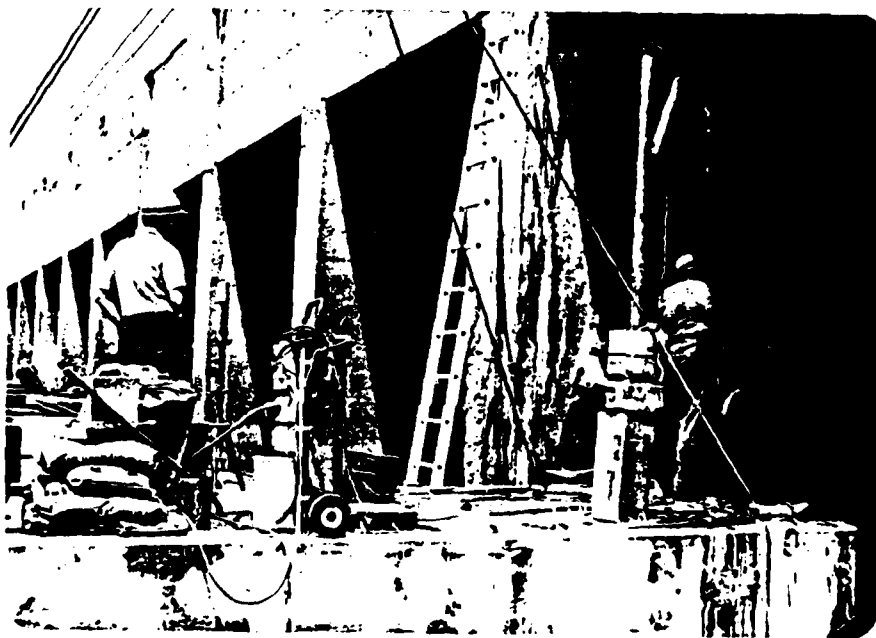


Figure 1b



(b) Set up of skid-rig on top of approach wall.



(a) Setup with portable drill on floating platform

Figure 2. Typical drill rig setups, Dresden Island.

15. Total footage drilled was 207.0, 103.7, and 49.6 ft in concrete, bedrock, and overburden, respectively. All concrete and bedrock samples were preserved for laboratory testing. Procedure for handling the core and preserving it for testing was the same as given in Reference 5. Color photographs of all the core recovered are included in a notebook as Exhibit A to this report. The Exhibit is on file at the Chicago District Office. Core recovery was very good in the concrete and bedrock. The recovery in the backfill material averaged 87 percent. The backfill material consists of sandy gravel and boulders and could not totally be recovered. All drilled holes were backfilled to their full depth with concrete as batched from a commercially available packaged dry combined mixture.

16. Two piezometers of the slotted 1-1/4-in. PVC pipe type were installed in two of the backfill holes. One piezometer was set at elevation (el)* 477.9 (piez, tip) in hole DI WES GWB-1-77 and another was set at el 478.4 in hole DI WES GWB-2-77. Plates 3 and 4 give pertinent data; daily drill reports on file at CDO contain measurements of water heights for the time interval the drill crew was on site.

17. Between el 470.1 and 468.8 in boring GW-2 the drill log indicated subangular gravel; the concrete-bedrock is at el 474. Two additional borings were put down within 15 ft of GW-2 in order to ascertain the extent of the gravel. The two borings were designated GW-8 (drilled through the approach wall) and GWB-3 (drilled in the backfill). The boring locations are given in Figure 1a.

18. The gravel was not found in either of these borings and it is therefore assumed that the gravel found in GW-2 was a local occurrence. Pieces examined in the laboratory were identified as subrounded limestone fragments partially coated with green gray clay; the material was called reworked limestone. The lens was encountered about 2.5 ft from the river side of the approach wall. It is assumed that undercutting has occurred due to the barge traffic. Should undercutting continue, the possibility exists for the limestone to be washed from beneath the wall.

* All elevations (el) cited herein are in feet referred to mean sea level.

PART III: GEOLOGICAL CHARACTERISTICS

Backfill

19. Three boreholes were made into backfill material (GWB-1, GWB-2, GWB-3) behind the lower approach wall; see Figure 1a for boring location. Approximately 90 ft of material was recovered. All three boreholes reached bedrock. Bedrock overburden was probably spoil from the lock and dam excavations and generally consists of a mixture of silty, clay, sand, gravel, and boulders. The backfill material varies in proportions from 60 percent clay on the surface (gravelly clay) to 60 percent gravel directly over bedrock (clayey gravel). Bedrock consists of 17 ft of Ft. Atkinson Limestone, a gray-green shale layer of variable thickness, and the dense gray Clermont Shale. Part of this sequence is missing in GWB-1 which has a 3-1/2-ft concrete layer between the clayey gravel layer and the gray shale layer. Borehole GWB-3 was drilled in the same area as GW-8 and GW-2. Contact of Ft. Atkinson Limestone and Clermont Member of the Scales Shale occurred at the same elevation (469 ft) in all three holes. It is therefore assumed that the thickness of the Ft. Atkinson Limestone at that point is 16.8 ft (thickness of limestone in GWB-3). Three hundred feet downstream along the approach wall are boreholes GW-5 and GWB-1 and 2. In borehole GWB-2 thickness of the Ft. Atkinson Limestone is 17.2 ft. The difference in elevation of the limestone: shale contact and of certain continuous dolomitic shale beds from one drilling area to the other is approximately 11 ft. Thus the dip of the bedrock beneath the approach wall is 2 degrees to the east which is consistent with available geologic information on the area.

Bedrock Stratigraphy

20. The bedrock beneath and near the dam is of Ordovician age and is assigned to the Ft. Atkinson Limestone and Scales Shale Formations of the Maquoketa Group.⁶ The Ft. Atkinson Limestone is a coarse-grained,

dense, crinoidal, pyritic limestone which contains many seams and pods of green clay and clayey shale. The limestone parts along clay and shale seams parallel to bedding. The Ft. Atkinson Limestone overlies a dense greenish to brownish grey shale which is the Clermont Member of the Scales Shale Formation. The shale is calcareous in part and contains seams of soft friable oxidized shale and relatively few fossils. There is one area in a calcareous section of the shale with a distinct layer of small fossils which may be the "depauperate zone" described in the literature on the Scales Shale (Templeton and William).⁷ The Clermont Shales slakes upon drying into discs 0.1- to 0.5-ft thick.⁴ The contact of the limestone with the gray shale is a thin green shaley clay layer which ranges in thickness from several inches to two feet.

Geologic Cross Section

21. A log of borings was drawn comprising three boreholes along the lower approach wall (GW-5, GW-2, and GW-8) and one (L-8) in Monolith 52 of the lower gate block recess. The log of borings is presented in Plate 5. Boring L-8 was drilled during the Phase II work and is included in this report for completeness. The logs were compiled to show an overview of the bedrock material as well as to assist in the selection of representative test specimens. A profile into bedrock for the backfill borings was not drawn because the bedrock beneath the backfill overburden is the same as found under the lower approach wall. A geologic cross section along section-A-A', including borings GW-5, GWB-2, GW-2, GW-8, and L-8 drilled during this investigation and SACDI-1 and SACDI-2 drilled during 1972 by the CDO, is presented in Plate 6. Slight differences in rock terminology can be noted between Plates 4 and 5. The reason for the difference is that Plate 4 was drawn using information given on field logs and Plate 5 was drawn using the same information but supplemented with data from X-ray examination. The term calcareous shale was used on the field logs, however, when this shale was X-rayed, it was found to be dolomitic shale.

22. A tight contact between concrete and bedrock was found in boring GW-8 while in the other two WES holes, the contact was loose.

The bedrock revealed by the three WES borings and two CDO borings has been subdivided into three units based on changes in lithology: light gray Ft. Atkinson Limestone, a green shaley clay at the contact between the Ft. Atkinson and Clermont units, and the dark gray Clermont Shale.

23. Approximately 60 ft of rock were received from the three borings in the lower approach wall; 10.5 ft of limestone, 4 ft of shaley clay, and the remainder of gray shale. The limestone is light gray with sticky green clay and green clayey shale seams up to 0.1-ft thick contained in it. It parts parallel to bedding along these seams. There are vuggy areas in the limestone as well as areas of vertical fractures, some healed. The thickness of the Ft. Atkinson Limestone beneath the lock as revealed by approach wall backfill borings into bedrock is on the order of 17 ft. Eight to ten percent of the gray Clermont Shale is dolomitic in sections 0.1-ft to nearly 2-ft thick, some containing fossils. The dolomitic portion of the shale is harder and more competent than the remainder.

24. Neither the limestone nor the green shaley clay is found in borehole GW-5, which is located farther down in the rock column than either GW-2 or GW-8. Correlation between GW-5 and GW-2 is made on the basis of positioning and repetition of dolomitic layers in the shale, especially the "depauperate zone" of fossils. This zone is located 17 ft below the Ft. Atkinson Limestone.

Bedrock Structural Characteristics

25. The bedrock structural characteristics relevant to foundations are presented in Plate 5 and represent the same material described under Geologic Cross Sections.

26. The bedrock is considered competent and massive below the Ft. Atkinson Limestone and clay layers. Compressive tests on the limestone in sections containing no clay seams proved the limestone to be extremely competent. Compressive tests on limestone containing the clay seams showed about one-half the strength as did the competent limestone. There was no dip apparent on the bedding surfaces of the cores, although

a local dip of 2 percent to the east was indicated by triangulation on a bed encountered in three boreholes. Several joints were encountered but were not continuous between cores.

27. The previous study done by the Corps of Engineers on Dresden Island states there is a normal fault one mile south of the lock site. It is described as minor in importance and inactive. The report also states that there were other normal faults found in borings adjacent to the lock, but there has been no movement along them for millions of years (since pre-Pennsylvanian time). For a more detailed analysis of these faults, consult the previous study.⁴

28. There was one stylolite found in GW-5. The oxidized seams in the Clermont Shale are all less than 0.1 ft in thickness, and are soft, very friable, and sometimes clayey. A thin gray clay layer was exposed at the bottom of the core from GW-5.

29. The data, bedrock stratigraphy, and structure contained in Reference 4 correlate quite well with similar data reported herein. Minor differences that do occur will not have any effect on the overall stability of the lower guide wall.

30. Possible weak zones in the bedrock under the lower guide wall are the reworked and fractured limestone found in GW-2 and GW-8, the clay seams in the limestone directly under the concrete in GW-2 and GW-8, and the green shaley clay layer which is continuous over the project. Atterberg limits for this green shaley clay layer and other foundation materials over the project are given in Reference 4.

PART IV: TEST SPECIMENS AND TEST PROCEDURES

Cores Received

31. About 284 ft of concrete and rock core along with eleven sacks of backfill material were received from a total of 41 borings. Pertinent information concerning the core received at WES is presented in Table 1. Upon receipt of the core at WES, the boxes containing rock samples were placed in a moist curing room until the selection of test specimens was completed. Selected test specimens were then stored in the same room until time for testing.

Selection of Test Specimens

32. The concrete core varied greatly in its physical condition over the project. Some core was so badly deteriorated (broken to gravel-size pieces) that intact samples could not be obtained for testing. Therefore, concrete test specimens were selected from the intact near surface portion and the bottom portion of the core. The near surface core sometimes contained cracks but remained intact. This procedure was used for the short core and for core with a maximum length of 5.2 ft. For the deeper borings, GW-2, GW-5, and GW-8 test specimens were selected from the near surface, middle, and bottom portions of the core. The specimen depths shown in the tables of test results represent the mid section of the test specimen; i.e., 0.5 ft is the mid-point of a piece of core from depth 0.0- to 1.0-ft. Characterization properties; effective (wet) unit weight (γ_m), compressional wave velocity (V_p), and compressive strength (UC), and engineering design properties; Young's Modulus (E), and Poisson's ratio (ν) were determined or calculated.

33. For the engineering design tests (modulus of elasticity, Poisson's ratio, and direct shear) an attempt was made to select test specimens to be representative of the rock in close proximity to the concrete-rock contact under the lower approach wall. Whenever possible bedrock specimens were selected for characterization properties tests

from the full length of core. The properties from the lower portions of the core were obtained in order to determine the quality of the rock in the zone where tendon anchors would be set. Where feasible, this was accomplished. The test assignment locations can be obtained from appropriate tables of test results.

34. There were four types of specimens tested in direct shear; concrete cast on rock, intact, precut, and cross-bedded. Adequate test specimens containing clay-filled partings or clay seams and natural joints could not be obtained. Limestone containing thin clay seams was received in the laboratory; however, the limestone was broken and specimens of appropriate length could not be prepared for direct shear testing. No natural joints were found in rock competent enough from which direct shear test specimen could be obtained. The majority of test specimens selected for direct shear were obtained within the first 10 ft below the concrete-rock contact.

35. Only three specimens of shale from boring GW-2 could be used for unconfined compressive testing, modulus and Poisson's ratio determination. Other pieces of core were of insufficient length. Four pieces of limestone core were adequate for compressive testing. Specimens for direct tensile strength were taken as close to the bottom of GW-2 as possible. Attempts to recover samples of the clay seams and green shaley clay will be made during the Phase II drilling program.

Test Procedures and Petrographic Examination

36. The characterization properties tests and the engineering design properties tests were conducted in accordance with the appropriate test methods tabulated below:

<u>Property</u>	<u>Test Method</u>
<u>Characterization</u>	
Effective Unit Weight (As Received), γ_m	RTM 109*

* Proposed Rock Test Method, Corps of Engineers, in review prior to publication.

<u>Property</u>	<u>Test Method</u>
<u>Characterization (continued)</u>	
Dry Unit Weight, γ_d	RTM 109
Water Content, w	RTM 106
Compressional Wave Velocity, V_p	RTM 110 (ASTM D 2845) **
Compressive Strength, UC	RTM 111 (ASTM D 2938)
Tensile Splitting Strength, T_s	ASTM C 496-71
<u>Engineering Design</u>	
Elastic Modulus	RTM 201 (ASTM D 2148)
Direct Shear Strength	RTM 203
Multistage Triaxial Strength	RTM 204

37. The concrete-on-rock specimens for direct shear testing were fabricated using a general mass concrete mixture having an approximate compressive strength of 2000 psi at 28 days age. The concrete was wet sieved over a 1-in. sieve size screen, and the portion passing was cast on top of rock cores contained in the bottom section of 6-in.-diameter molds. Rock surfaces onto which the concrete was cast were quite flat. Rock cores used for these tests were taken from within 3 ft of the dam concrete-rock contact.

38. A detailed visual examination of all the core was made in the laboratory. Pieces of concrete core were selected for petrographic examination from all structures of the lock and dam that were drilled. These pieces of cores were sawed along the long axis of the core to allow better and easier examination of the concrete. Some of the new surface concrete recovered were highly fractured and examination was made of the pieces. A photograph was made of the sawed surfaces of each condition represented by the cores. The concrete cross-section was examined megascopically and with a stereomicroscope. Selected particles were examined using a polarizing microscope and identified using x-ray diffraction. Portions of the cement paste in a deteriorated area of concrete was compared to the cement paste from an area of concrete that was considered good.

39. A representative sample of green clay was ground to pass a 45 micrometer (No. 325) sieve and then examined by x-ray diffraction. A sedimented slide was made in the usual manner and also x-rayed. A

dried portion of the sample was immersed in water to observe any slaking properties. The x-ray examinations were made with a x-ray diffractometer using nickel-filtered copper radiation.

PART V: TEST RESULTS AND ANALYSIS

Bedrock Under Lower Approach Wall

Results of petrographic examination

40. A piece of greenish gray (SGY 6/1)⁸ clay recovered from GW-2, el 469.5 was analyzed to determine its mineralogical composition. The rock after being air dried slaked heavily when immersed in water. It was composed of quartz, dolomite, siderite, plagioclase, feldspar, and muscovite clay. The clay is identified as a clay-mica and is non-swelling. The other rocks under the lower approach wall were visually identified; they are described in Part III.

Characterization properties

41. The results of the characterization properties tests of the bedrock are presented in Tables 2 and 3. The average value, the range (difference between highest and lowest values), and the number of tests for the bedrock are tabulated below:

<u>Summary of Bedrock Characterization Properties</u>						
	<u>γ_m, lb/ft³</u>	<u>γ_d, lb/ft³</u>	<u>w, %</u>	<u>Vp, Fps</u>	<u>UC, psi</u>	<u>T_s psi</u>
<u>Limestone</u>						
Average	176.5	175.0	0.9	18,004	10,340	
Range	2.5	2.1	0.5	3,885	7,180	
No. of Tests	4	4	4	4	4	
<u>Shale</u>						
Average	155.4	144.8	6.9	8,778	1,230	195
Range	6.9	5.8	3.2	940	570	50
No. of Tests	3	3	6	3	3	3

42. The unit weights of the limestone are consistent and reasonable. The average γ_m is 176.5 lb/ft³ which is a good bit higher than a similar value reported in Reference 4, i.e., 155 lb/ft³. The average moisture content is reasonable.

43. The average Vp for the limestone is 18,004 fps and is considered reasonable for dense limestone. The previous reported value obtained on 1-in.-diameter by 2-in. specimens is 9400 fps. Reference 4

does not specify specimen condition including water content; therefore, no explanation for the large difference in V_p is offered.

44. The average unconfined compressive strength of the limestone was obtained for specimens having 6-in. diameters and 12-in. lengths. Some of the specimens contained thin shaley clay or clay seams. These features did affect the UC as seen in Table 2. The high UC of a specimen from GW-2 (15,170 psi) represents a more competent zone of the limestone that was found in borings GW-8 and GW-2. The average UC is 10,340 psi and when compared to the UC value of 3000 psi reported in Reference 4, is quite high. Reference 4 does site sample preparation difficulties which could account for some of the differences in strength.

45. The unit weight of the shale are reasonable. The average γ_m is 155.4 lb/ft³ and compares quite well with the previously reported values of 150.0 lb/ft³. These values indicate a moderately dense, compact shale.

46. The V_p obtained on the shale perpendicular to bedding averaged 8778 fps which compares fair with the previous average value of 6126 fps. The average unconfined compressive strength of the shale was obtained on 6-in.-diameter by 12-in. long cylindrical specimens; the average UC is 1,230 psi with a range of 570 psi. The relatively large sample size generally gives more realistic strength values than does the smaller size (1-in. by 2-in.) as reported in Reference 4. They report an average UC of 5600 psi; this value is highly questionable as being representative of the shale directly under the lock walls. Again, no specific information regarding specimen condition or water content was given in the report; therefore, an explanation of the strength difference for the shale is not offered. A check was made to see if the UC specimens reported previously and herein were obtained from about the same elevation and were the same type shale. As near as could be determined, they represent the same shale.

47. Tensile strengths were determined for the shale; adequate limestone specimens were not obtainable. The average tensile splitting strength for the shale is 195 psi.

Engineering Design Properties

Modulus of elasticity and Poisson's ratio

48. Results of the modulus of elasticity and Poisson's ratio tests are presented in Tables 2 and 3. The stress-strain relation for the limestone and shale are presented in Plates 7 and 8, respectively. The E was calculated as an incremental value between 0 to 500 psi, in most cases the 0 to 500 psi increment corresponded to the initial linear portion of the stress-strain curve for both rocks.

49. The average E for the limestone and shale is 5.51×10^6 psi and 0.25×10^6 psi, respectively. These values are reasonable. Reference 4 reports a E for the limestone equal to 0.313×10^6 psi; this value is highly questionable. The average Poisson's ratio for the limestone and shale is 0.25 and 0.36, respectively.

Multistage triaxial test

50. A multistage triaxial test was conducted using pieces of concrete and shale from boring GW-5. The test results are questionable due to a leak in the membrane. It is recommended that another multistage test be run during the Phase II drilling and testing program.

Peak shear strength

51. The stress value for the direct shear tests are presented in Table 4; the direct shear envelopes are plotted on Plate 9.

52. Two types of direct shear tests were conducted to ascertain peak strength of intact specimens and sliding friction characteristics of specimens containing discontinuities. Peak strengths were measured for the shale containing a concrete-rock interface, intact shales, and cross-bedded intact shales. Precut specimens of shale were tested for sliding friction.

53. All specimens were tested in the single-plane shear device designated the MRD shear device. The tests performed on intact, concrete-on-rock, and cross-bedded specimens produced a moderate amount of scatter. All envelopes were calculated using a linear regression analysis.

54. Six concrete-on-rock tests were conducted along the interface of concrete and rock, three tests with limestone and three tests with

shale. All specimens contained the natural rock bedding planes. The peak shear strength is quite different between the concrete to limestone and the concrete to shale. The c and ϕ is 19.05 tsf and 67.2 degrees, respectively, for the limestone specimens. The c and ϕ is 5.56 tsf and 49 degrees for the shale specimens. These shear strength values are considered reasonable. The ultimate shear strength parameters for the concrete-on-limestone indicate a large reduction in shear resistance; the ultimate $c = 0.0$ tsf and the residual ϕ_u is 37 degrees. The ϕ_u of 37 degrees approaches the coefficient of friction normally obtained on precut concrete-on-limestone specimens (28 to 32 degrees). The ϕ_u for the concrete-on-shale is 49 degrees or equal to the ϕ obtained from the peak shear strengths.

55. The shear strength parameters for the intact shale are $c = 3.56$ tsf and $\phi = 47.2$ degrees. These values were obtained from specimens tested parallel to bedding. The values in Reference 4 compare fairly well; i.e., $c = 2.30$ tsf and $\phi = 49.5$ degrees. The ϕ_u for the intact shale tested during this study is 42.0 degrees and compares quite well with the ϕ of 49.5 degrees reported in Reference 4.

56. The cross-bedded specimens were tested at an angle of 45 degrees to bedding. The c and ϕ is 56.6 tsf and 46.3 degrees; the ϕ_u is also 46.3 degrees. These values from the cross-bedded shear tests were recommended to be used in the stability analysis for that portion of the bedrock at the toe of the lower approach wall.

57. The precut specimens consist of shale on shale. Two multi-loading tests were conducted on the green gray shale and one on a dolomitic piece of the same shale. The ϕ is 17 percent greater for the dolomitic shale. The recommended c and ϕ for the precut shale is 0.1 tsf and 21.4 degrees. The ϕ_u is slightly less than the peak ϕ ; i. e., $\phi_u = 20.2$ degrees. Refer to Table 4 for summary of peak and ultimate shear strength parameters.

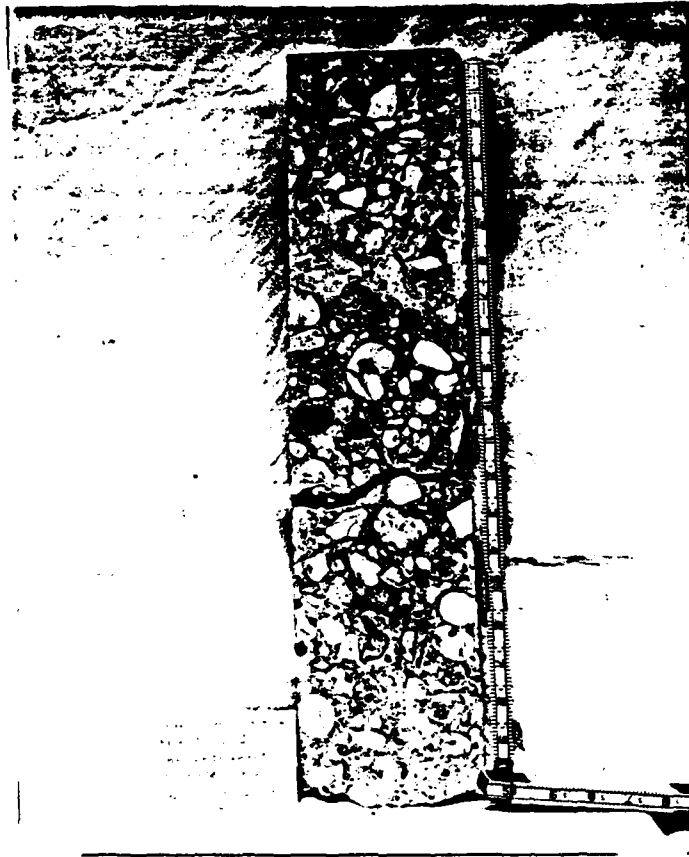
General

58. The following general comments pertain to the condition of the concrete over the entire lock and dam. Individual structures within

the lock and dam will be discussed separately. The results of the concrete characterization and engineering design tests are presented in Table 5. These data will be referred to when appropriate. General description of the concrete from 34 borings are presented in Plates 10-15; a description of the cores examined in detail is presented in Plates 16-24. The field drilling logs for all borings except those on the inside faces of the lock walls are presented in Exhibit B to this report. Exhibit B is on file at the Chicago District Office. The reader is referred to Reference 2 for excellent photographs depicting typical exposed concrete surfaces showing severe erosion and scabbing of concrete.

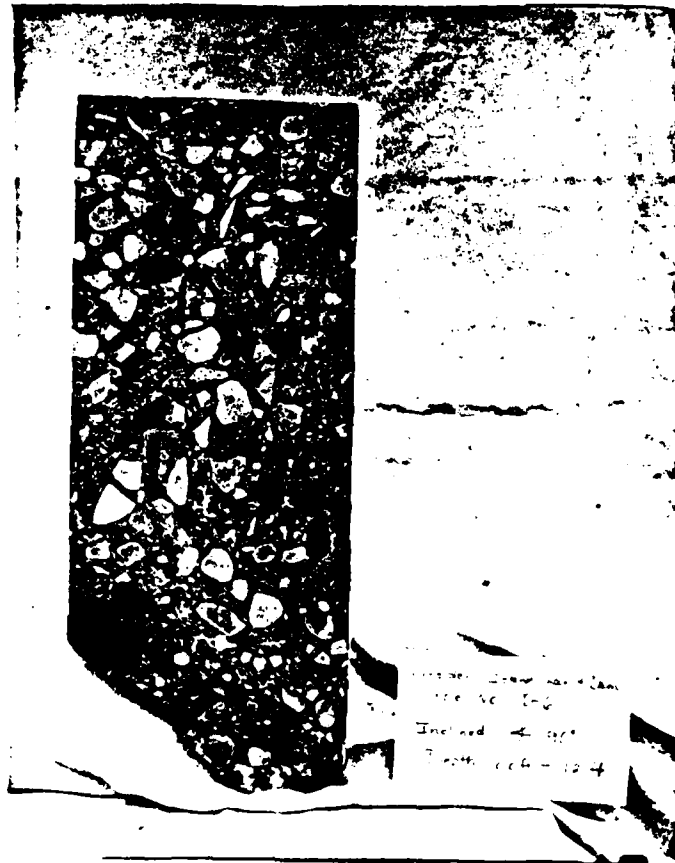
59. All of the concrete examined except for the near surface concrete in borings D-1, D-6, GW-1, GW-2, GW-5, and GW-8 did not contain any entrained air. The non-air entrained concrete was considered old concrete and the air entrained concrete was considered new. The new concrete normally consisted of coarse aggregates less than 1 in. in diameter and was cemented with a light gray paste. The old concrete consisted of coarse aggregate with a maximum size about 3 in. in diameter and was cemented with a brown paste. The concrete was normally composed of a natural carbonate coarse aggregate with occasional igneous rock and chert particles and a natural siliceous fine aggregate. The concrete in all cases was well consolidated and contained some entrapped air. No large areas of honeycombing were detected in the borings; occasional fist-size areas were seen.

60. The concrete varied in physical condition. The new concrete was intact and in good condition but as shown in Figure 3 the old concrete in boring D-1 is fractured to a depth of 1.5 ft. This depth is about 0.7 ft below the contact of the new and old concrete. In boring D-6, as illustrated in Figure 4, and in boring GW-5 a crack exists about 0.1 ft deeper than the contact of the old and new concrete. In boring GW-2 an old crack exists at the contact of the old and new concrete. Borings GW-1 and GW-8 were the only other borings with new concrete overlying the old concrete. The concrete was intact and in good condition throughout the entire length of core.



New concrete ends at about 0.8 foot depth and old concrete is below it. Parallel to subparallel fractures are to a depth of about 1.5 feet.

Figure 3. New and Old Concrete



New concrete is to a depth about 1 foot. The break parallel to the contact of the old and new concrete was considered to be old.

Figure 4.

61. The near surface old concrete in some areas was intact and did not show any signs of deterioration. Other areas were highly fractured but remained relatively intact as illustrated in Figures 5 and 6. In other areas the surface had scaled off and the concrete near the surface was not intact as shown in Figure 7.

62. The depths of deterioration shown in Plates 10-15 are caused by frost action. This deterioration of the concrete was characterized by cracking parallel and subparallel to the surface of the concrete. The cracks propagate through aggregate as well as the paste. The depth of frost damage of the concrete was indicated at the deepest point when cracking was through the aggregate and paste.

63. Some of the fractures were filled with a white precipitate. This white precipitate consisted of calcite, gypsum, and ettringite. Ettringite (calcium sulphoaluminate hydrate) is formed in concrete when calcium sulphate and calcium aluminate (mainly C_5A_3) reacts in the presence of water. It is believed, however, that this product developed after the cracking had already formed and was not the cause of the deterioration.

Lock Concrete Condition

Lock chamber land wall

64. Depth of deterioration. The average depth of concrete deterioration through the eroded portion of the wall is 0.7 ft; the maximum depth is 1.2 ft (see Figure 8 for depth of deteriorated concrete for each hole). Figure 8 is a duplicate of Plate 1 which shows both boring locations and depth of deterioration; it is presented here for continuity. The total depth of deterioration as measured from the original face of the lock wall is the sum of the eroded depth and of the present frost damaged concrete. The deepest damaged concrete is near the top of the wall.

65. Average physical properties. The average physical properties of the land wall concrete are tabulated below:



The parallel and subparallel fractures decrease in frequency as the depth increases. The near surface fractures have been wedged apart by frost action and is easily removed.

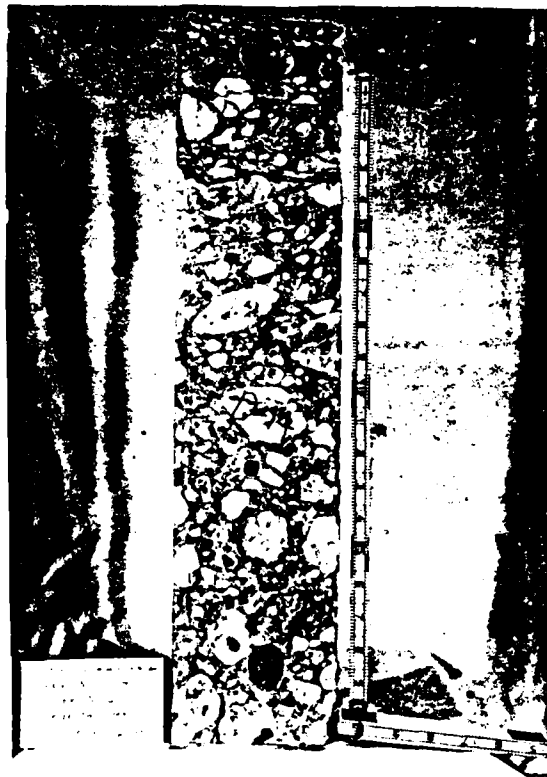
Figure 5. Effects of frost action



Some of the cores were not intact. The surface had scaled off and the pieces were easily separated.

Figure 7. Severely damaged concrete.

Dresden Island Lock and Dam
Hole No. D-14
Pier No. 6
Horizontal Core
Depth 0.0 ft - 23 ft

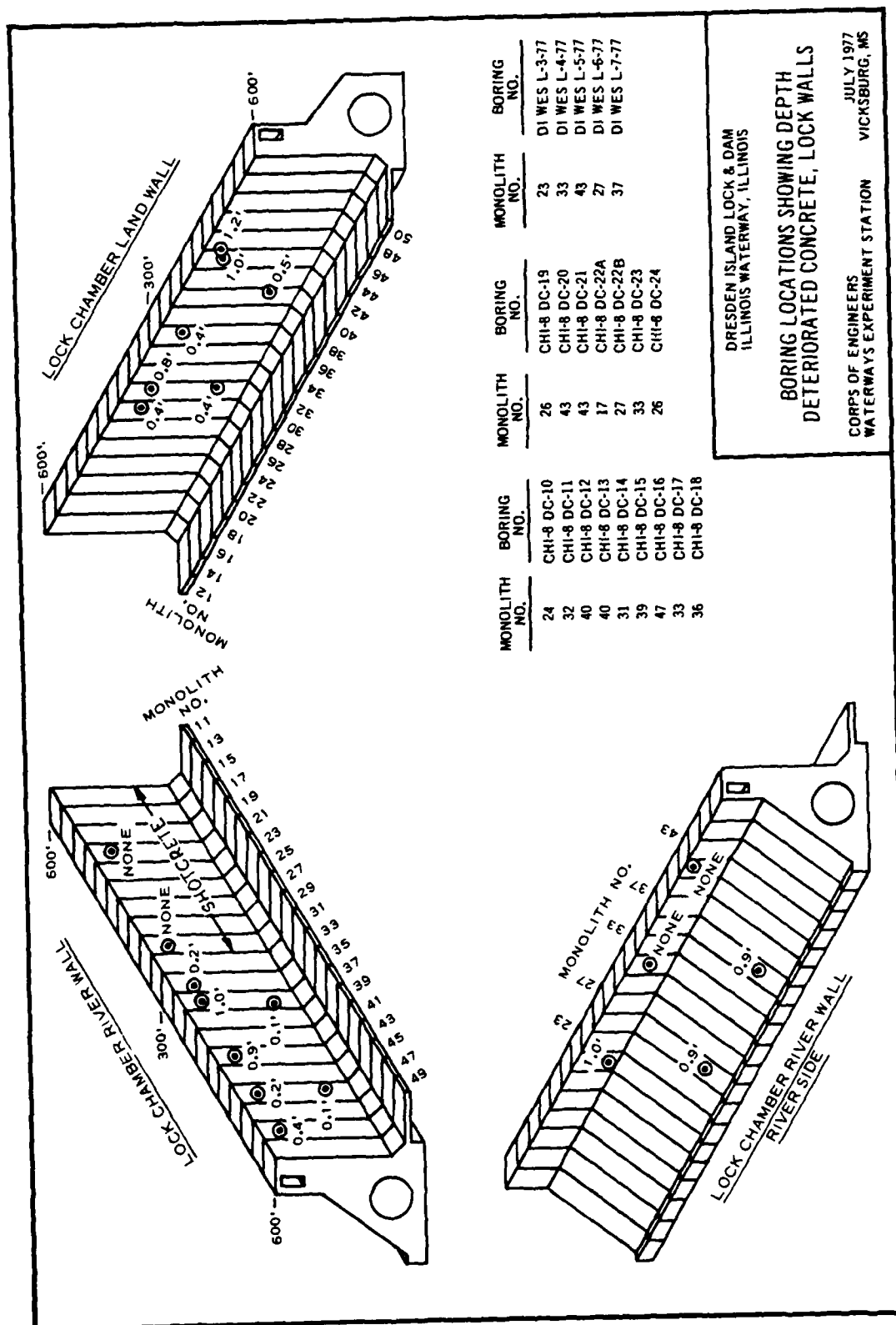


Not all of the fractures were caused by frost action. Some of the fractures were caused by poor consolidation as shown by vertical crack near the bottom of this piece of core.



A close up of D-14 shows fractures going through paste and extending into aggregates. The near surface pieces were easily removed.

Figure 6. Effects of frost action.



DRESDEN ISLAND LOCK & DAM
ILLINOIS WATERWAY, ILLINOIS

BORING LOCATIONS SHOWING DEPTH
DETERIORATED CONCRETE, LOCK WALLS

CORPS OF ENGINEERS
WATERWAYS EXPERIMENT STATION

JULY 1977
VICKSBURG, MS

Figure 8

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	149.7	150.0
Comp Wave Velocity, fps	13,438	15,258
Compressive Strength, psi	5,840	5,860
Modulus of Elasticity, x 10 ⁶ psi	2.75	4.57
Poisson's Ratio	0.13	0.21

Lock chamber river
wall, chamber side

66. Depth of deterioration. The average depth of deteriorated concrete throughout this side of the wall is 0.7 ft; the maximum depth is 1.0 ft (see Figure 8 for depth of deteriorated concrete for each hole). The deepest frost damaged concrete is near the top of the wall. All of the cores exhibit frost damage near the vertical lock wall surface.

67. Average physical properties. The average physical properties of the river wall concrete are listed below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	150.2	149.7
Comp Wave Velocity, pcf	13,075	15,575
Compressive Strength, psi	5,060	5,430
Modulus of Elasticity, x 10 ⁶ psi	2.75	4.57
Poisson's Ratio	0.15	0.20

Lock chamber river
wall, river side

68. Depth of deterioration. The average depth of concrete deterioration in borings L-3, L-6, and L-7 is 0.9 ft; the maximum depth is 1.0 ft. Frost damage was not evident in borings L-4 and L-5 (see Figure 8 for location). It is possible that the areas where no frost damage showed up in the cores, the concrete was well consolidated and the surface not cracked so that water could penetrate the concrete. Severe erosion and scabbing of the concrete is occurring in many locations along the wall.

69. Average physical properties. The average physical properties of the river wall concrete are tabulated below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	146.7	148.1
Comp Wave Velocity, fps	13,833	14,659
Compressive Strength, psi	5,580	5,380
Modulus of Elasticity, $\times 10^6$ psi	2.90	4.18
Poisson's Ratio	0.18	0.22

Upper gate bays

70. Depth of deterioration. The average depth of deterioration in the gate bays is 1.5 ft; the maximum depth is 1.6 ft in boring L-1 which was drilled into the riverward upper gate bay (see Figure 1b for location of borings). This extent of damaged concrete in a location subjected to high stress, such as a gate bay, is considered extremely dangerous.

71. Physical properties. Only one specimen from L-2 was suitable for testing. The results of the velocity test is 13,861 fps which is in the lower range of values obtained for all the cores. The value was measured at a depth of 2.5 ft; most all the other values of velocity at this depth show velocities in the 14,000 to 16,000 fps range. The compressive strength was 4,460 psi.

Upper approach wall

72. Depth of deterioration. There was no frost damaged concrete detected in the one core taken from the upper approach wall. The core consisted of new concrete and under a detailed examination appeared in excellent condition.

73. Physical properties. Only one piece of core was tested. The density, velocity, strength, and modulus represented sound concrete. The compressive strength of 7840 psi was the highest value obtained over the project; however, it represents concrete not effected by freezing and thawing. Severe erosion is evident at the water line and at monolith joints.

Lower approach wall

74. Depth of deterioration. There were three vertical holes drilled into the lower approach wall and four horizontal holes. The core from

the vertical holes showed 0.5-ft to 1.5-ft of new concrete. The new concrete was characterized by less than 1-in. maximum size coarse aggregate, a light gray paste; and it was air entrained. There was no apparent frost damaged concrete detected in the three vertical holes.

75. There was no apparent frost-damaged concrete in the highest horizontally drilled holes; they are located about 7.2 ft from the top of the wall. Although the core from one of these borings (GW-6) contained a longitudinal crack extending the full 3.4 ft of the core, no evidence of frost damage was seen. The crack was coated with algae and other debris. It is not known if the crack runs vertically or horizontally in the wall.

76. The average depth of damaged concrete in the lowest horizontally drilled holes (11 ft below top of wall) is 1.5 ft; this corresponds to the maximum depth of bad concrete. Figure 9 presents details of the damaged concrete in the lower approach wall. The vertical borings were 3.0 ft back from the face of the wall and did not encounter the damaged concrete observed in the core from the lowest borings. It is assumed that the damaged concrete is present in a zone from 7.2 ft below top of wall to a foot or so below mean lower pool and 1.5 ft into the wall. Severe erosion was noted at the monolith joints and at the water line.

77. Average physical properties. The average physical properties of the lower approach wall are presented below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	145.5	148.9
Comp Wave Velocity, fps	12,500	14,357
Compressive Strength, psi	4,870	4,370
Modulus of Elasticity, $\times 10^6$ psi	4.55	3.49
Poisson's Ratio	0.27	0.21

The lower velocity of the near surface concrete reflects the subparallel cracking in the concrete (see Figure 6 for example). However, the strength of the same core does not indicate the damaged concrete. The reason is that often times cracks in cores that are perpendicular to the axis of applied load have little effect on compressive strength.

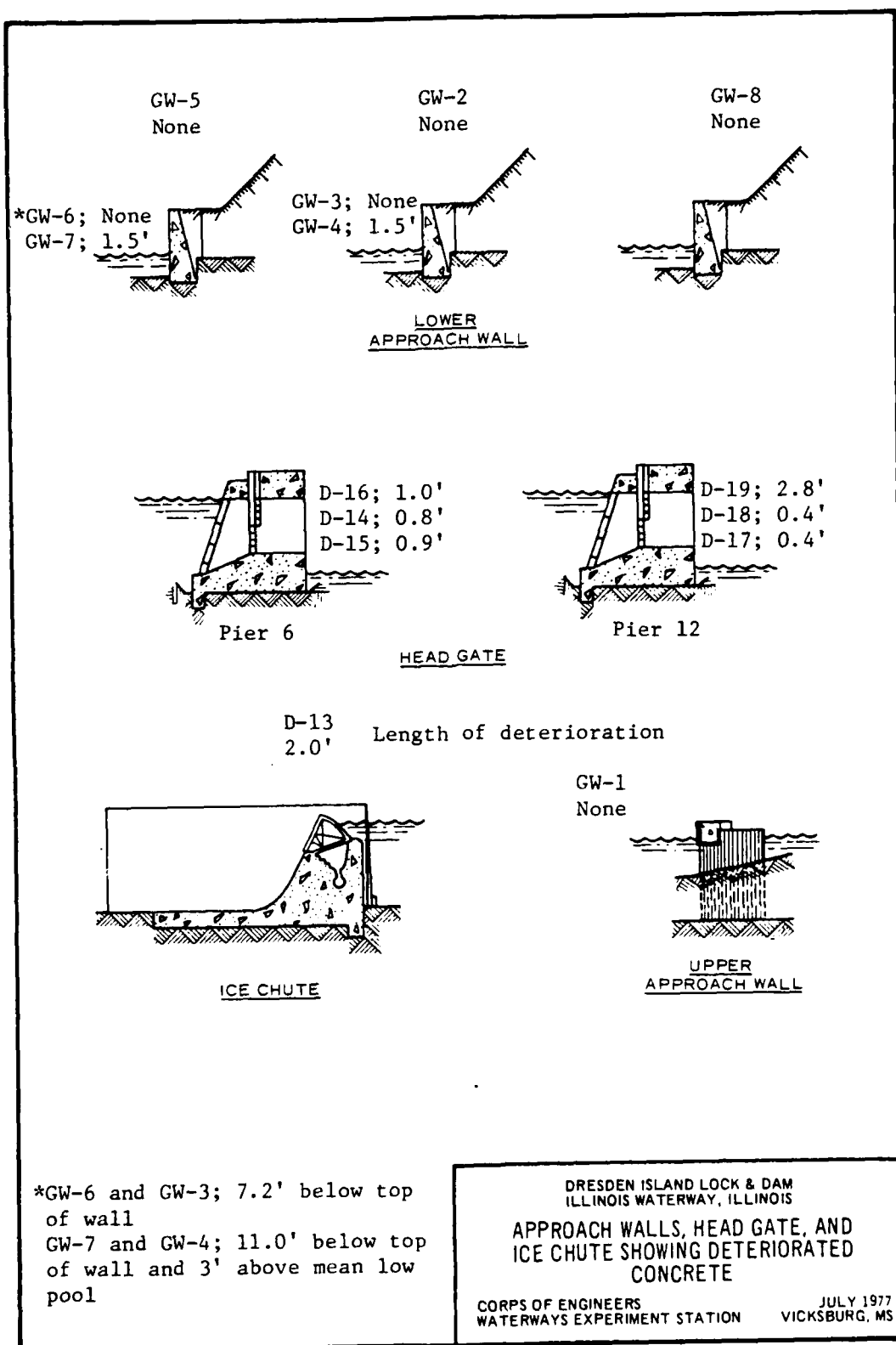


Figure 9

Dam Concrete Condition

Arch dam

78. Depth of deterioration. The depth of frost action observed in the core from the guard walls and the abutments of the arch dam are presented in Plate 12. Boring D-2 did not have any frost damage and boring D-3 only had minor frost damage to 0.2-ft depth. D-2 and D-3 borings were taken in the downstream portions of the abutment. Boring D-1 was capped with about 0.7 ft of new concrete that was not damaged; however, an additional 0.7 ft of old concrete beneath the new concrete was frost damaged.

79. The average depth of concrete deterioration in the guard wall on the arch dam abutments, including the side facing the spillway dam, is 1.3 ft; the maximum depth is 1.5 ft. Severe erosion is present at the water line and at monolith joints. The 1.5 ft of damaged concrete in the guard walls gives cause for concern; weakening of the concrete with continued deterioration could contribute to local structural failures.

80. Average physical properties. The average physical properties of the arch dam guard walls and the abutment are presented below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	147.3	147.3
Comp Wave Velocity, fps	13,076	13,940
Compressive Strength, psi	3,770	5,350
Modulus of Elasticity, $\times 10^6$ psi	2.75	4.52
Poisson's Ratio	0.15	0.23

These results show a marked contrast in strength, modulus, and Poisson's ratio. The near surface concrete is a good bit weaker than the deeper concrete. Similar data given in Reference 10 correlate quite closely with the above.

Tainter gate piers

81. Depth of deterioration. No damaged concrete caused by frost action was detected in the three borings drilled into pier No. 3; see Figure 10. However, there were signs of erosion on the sides of the pier

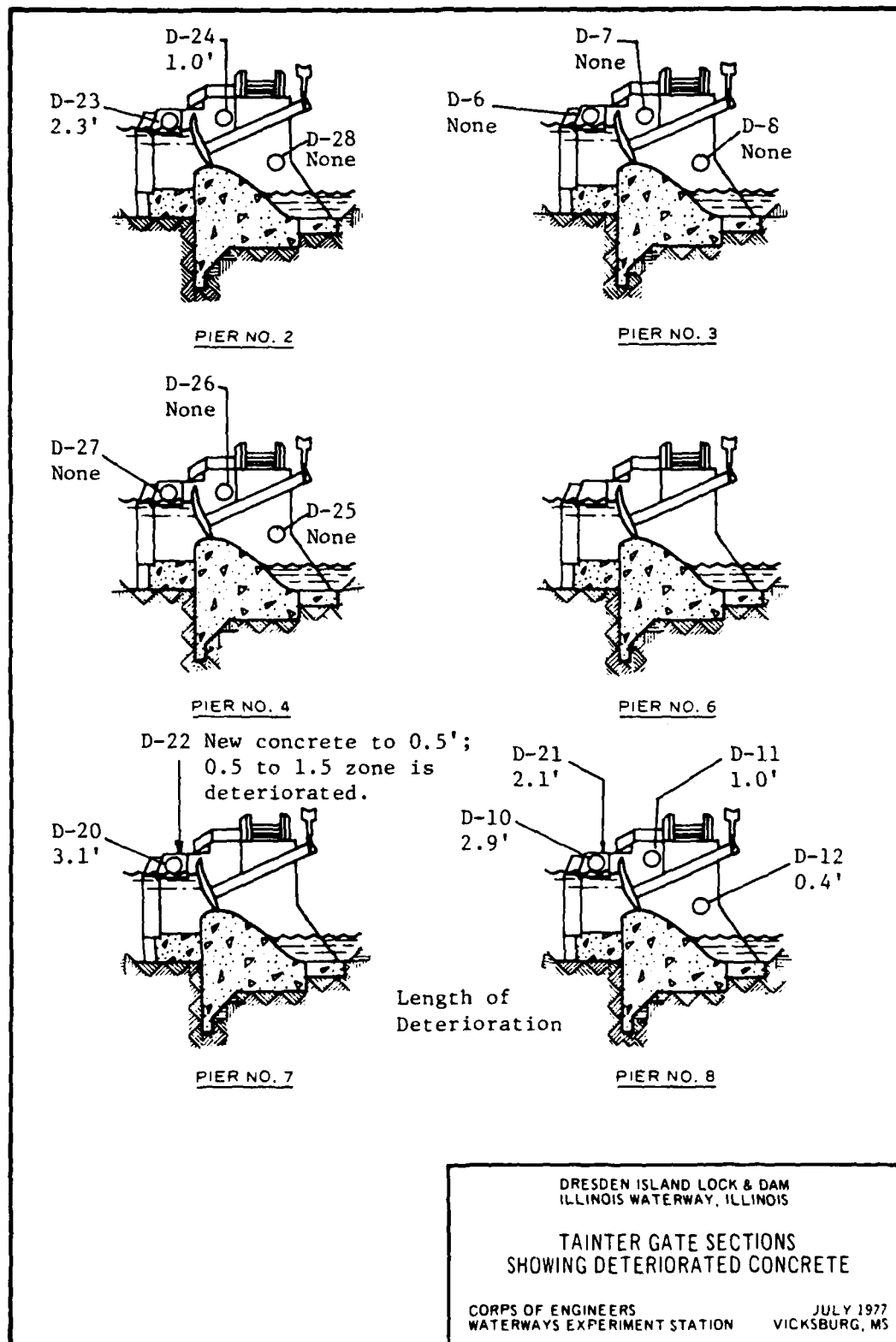


Figure 10

as reported in Reference 2. Additional borings were drilled in tainter gate piers 2, 4, 7, and 8 to more fully determine the extent of bad concrete in this section of the dam. Extensive deterioration was found in the core from the upstream one-half portion of piers 2, 7, and 8. No evidence of frost damage was found in pier 4.

82. The average depth of deterioration in the upstream one-half portion of pier 2 is 1.7 ft; the maximum depth is 2.3 ft. The average depth of deterioration in that portion of piers 7 and 8 from about 10 ft in front of the tainter gate and upstream to the nose of the pier is 2.3 ft; the maximum depth is 3.1 ft in pier No. 7, boring D-20. It appears that the greatest amount of frost damaged concrete in the tainter gate piers occurs in the upstream one-half portion and above mean upper pool elevation. Severe erosion has occurred at the upstream water line around the nose of the piers. Up to 0.4 ft of damaged concrete was found in boring D-12 located near the downstream edge of pier No. 8 (see Figure 10); no damaged concrete was detected near the downstream edge of piers 2 and 4.

83. Average physical properties. The average physical properties of the tainter gate piers are tabulated below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	148.6	149
Comp Wave Velocity, fps	15,593	15,000.0
Compressive Strength, psi	4,940	4,470
Modulus of Elasticity, $\times 10^6$ psi	4.76	4.31
Poisson's Ratio	0.21	0.19

The physical properties indicate sound concrete at selected locations in the piers; however, the badly deteriorated concrete could not be tested. The concrete not effected by frost action is structurally sound. It is realistic to say that most of the exposed concrete in the piers is deteriorated to some extent, a reasonable estimate would be 1.0 ft.

Ice chute pier

84. Depth of deterioration. Only one vertical boring was drilled into the ice chute pier. The maximum depth of frost action is 2.0 ft. The boring is located about 10 ft from the upstream edge of the pier and next to the oil storage tank located on the pier.

85. Physical properties. The physical properties of core from one boring in the ice chute pier are listed below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	146.7	146.7
Comp Wave Velocity, fps	13,513	13,698
Compressive Strength, psi	4,350	5,130
Modulus of Elasticity, $\times 10^6$ psi	2.62	3.88
Poisson's Ratio	0.15	0.21

Erosion is evident at the upstream end of the pier near the water line.

Head gates

86. Depth of deterioration. The average depth of deteriorated concrete in the head gate piers and sills is 1.0 ft; the maximum depth is 2.8 ft. See Figure 9 for damaged concrete at each of the boring locations. The greatest frost action has occurred on the downstream face of the piers just under the overhead slab.

87. Average physical properties. The average physical properties of the head gate concrete are tabulated below:

<u>Test</u>	<u>Near Surface Concrete</u>	<u>Bottom of Core Specimens</u>
Effective Unit Wt, pcf	149.2	146.5
Comp Wave Velocity, fps	12,550	15,185
Compressive Strength, psi	4,690	4,950
Modulus of Elasticity, $\times 10^6$ psi	1.86	4.11
Poisson's Ratio	0.13	0.22

The velocities, moduli, and Poisson's ratios indicate a large difference in the quality of the concrete in the head gate piers and sills. Again the strength is not necessarily a good indicator of quality for this type of damaged concrete.

PART IV: SUMMARY OF BEDROCK AND CONCRETE CONDITION
AND RECOMMENDED DESIGN VALUES FOR ROCK

Lower Approach Wall Bedrock Condition

88. Two stratum were encountered beneath the lower approach wall; the limestone of the Ft. Atkinson Formation and the shale of the Scales Formation. Both formations are in the Maquoketa Group. The limestone is a coarse-grained, dense, crinoidal, pyritic limestone which contains many seams and pods of green clay and clayey shale. The shale is dense, greenish to brownish gray, dolomitic in part, and contains seams of soft friable shale; it is the Clermont Member of the Scales Shale Formation. The contact of the limestone with the gray shale is a thin green shaley clay layer which ranges in thickness from several inches to two feet. The shaley clay layer is present under the wall and continuous under the lock and dam site.⁴

Geologic cross-section

89. A tight contact between concrete and bedrock was found in hole GW-8 while in the remaining two holes, contact was loose. The cross-section gives an overview of the bedrock material and depicts the clay and clayey shale seams up to 0.1-ft thick that occur in the limestone. The limestone contains vuggy areas and vertical fractures. The shale is fossiliferous and dolomitic in sections 0.1-ft to nearly 2-ft thick. The dolomitic shale was used to correlate the strata and give information concerning approximate dips of the bedding.

Bedrock structural characteristics

90. The bedrock under the lower approach wall is considered competent and massive below the Ft. Atkinson Limestone and the green shaley clay layer. Clay seams occur frequently enough throughout the limestone to negate any overall competency (in terms of shearing resistance) in the unit. A local dip of 2 degrees to the east was indicated by triangulation on a bed encountered in three boreholes. A few joints are present but were not continuous between borings.

91. The oxidized seams in the Clermont Shale are soft, very friable, and sometimes clayey. A thin clay layer was observed in boring GW-5 at

about el 449.8. No active faults are reported near the project site although a normal fault is reported to be located 1 mile south of the lock.⁴

92. Possible weak zones in the bedrock under the lower approach wall are the reworked and fractured limestone found in GW-2 and GW-8 and the clay seams in the limestone directly under the concrete in GW-2 and GW-8. The green shaley clay layer is continuous over the project. Failure in the bedrock would be expected to occur by sliding along clay seams in the limestone or in the shaley clay layer directly under the limestone. Pieces of the shaley clay that were recovered were of insufficient length for direct shear testing. A few samples were obtained during the second phase, and these samples will be tested for direct shear strength.

Backfill

93. The backfill behind the lower approach wall is probably spoil from the lock and dam excavations and generally consists of a mixture of silt, clay, sand, gravel, and boulders. Samples from the backfill could not be tested as originally planned because no undisturbed samples could be taken.

Recommended Design Values for Rock

94. Design should consider rock type and the bedrock structural characteristics described herein. Guidance is presented in the following tabulation as to proper choice of design parameters:

	<u>Limestone</u>	<u>Shale</u>
Characterization Properties		
Dry Unit Weight, lb/ft ³	175.0	144.8
Wet Unit Weight, lb/ft ³	176.5	155.4
Bearing Capacity, tsf	744	88
Tensile Strength, psi	--	195
Shear Strength		
Intact	--	c=3.6 tsf $\phi=47.2^{\circ}$
Clay-Filled Parting	--	--
Precut, Rock-on-Rock	--	c=0.1 tsf $\phi=21.4^{\circ}$

	<u>Limestone</u>	<u>Shale</u>
Concrete on Rock	c=19 tsf $\phi=67.2^{\circ}$	c=5.6 tsf $\phi=49^{\circ}$
Cross-Bed		c= 5.7 tsf $\phi=46.3^{\circ}$
Modulus of Elasticity, 10^6 psi	5.51	0.25
Poisson's Ratio	0.26	0.36
Shear Modulus, 10^6 psi	2.19	0.10

Lock Concrete Condition

95. The new concrete encountered in four of 15 borings is in good condition; the concrete occurs as a capping over old concrete. It is air entrained and has resisted the harsh winter environment. The new concrete is structurally sound by itself but in certain locations could be knocked loose by barge impact because of the frost damaged concrete beneath. The old concrete in the lock structures is non air entrained concrete that was well consolidated during placement. It is structurally sound in areas which have not been effected by frost action.

96. The exposed and near surface old concrete in the lower approach wall, lock walls (except for the gunite)² and upper gate bays is light to severely deteriorated. About 80 percent of the exposed vertical surfaces of the lock concrete have been effected by frost action to varying degrees. The result of frost action is evidenced by erosion and scabbing of the concrete. The average depth of concrete deterioration is as follows: lock chamber land wall, 0.7 ft; lock chamber river wall, 0.7 ft; lock chamber river wall, river side, 0.9 ft; and the upper gate bays, 1.5 ft. Immediate repair in the upper gate bays is necessary for the safety of the lock structure. Severe erosion of concrete is evident at most monolith joints and at the mean water lines.

97. It is recommended that 9 to 12 in. of concrete on exposed vertical and sloped surfaces be removed and replaced with new concrete. Localized areas may require deeper removal.

Dam Concrete Condition

98. The new concrete encountered in three of 19 borings is similar to that described under Lock Concrete Condition. The old concrete in the dam structures is non air entrained concrete and structurally sound in areas which have not been affected by frost action.

99. The exposed and near surface old concrete in the future lock walls and abutment of the arch dam, and head gate piers and sills is moderately to severely deteriorated. Frost action is the primary cause of the deterioration. Erosion and scabbing of the concrete is in evidence on most of the dam concrete. Erosion is more prominent near mean upper and lower pool elevations, while scabbing occurs on most of the concrete surfaces.

100. The average depth of concrete deterioration is as follows: arch dam future lock walls, 1.3 ft; arch dam abutment, 0.2 ft; spillway dam abutment, 1.0 ft; ice chute pier, upstream one-third of pier, 2.0 ft; head gate piers and sills, 1.0 ft.

101. Tainter gate pier No. 8 was selected to represent the five piers adjacent to the ice chute (piers No. 6, No. 7, No. 8, No. 9, and No. 10); these piers are severely deteriorated over 90 percent of their exposed surfaces. The average depth of concrete deterioration in the upstream half of these piers is 2.3 ft; a maximum depth of 3.1 ft was observed in pier No. 7. Deterioration in the remaining portion of the piers appears to be to a depth of about 0.4 ft.

102. Tainter gate pier No. 3 was initially selected to represent piers No. 1 through No. 5; these piers are visually moderately to severely deteriorated. The cores from pier No. 3 showed no damage due to frost action; however, erosion at the water line and scabbing of the concrete is present. Piers No. 2 and No. 4 were later drilled to verify the absence of frost damaged concrete in piers No. 1 through No. 5. Pier No. 4 revealed no frost damaged concrete while pier No. 2 showed an average depth of damaged concrete to 1.7 ft in the upstream one-half of the pier. The greater majority of exposed concrete at the lock and dam contains about 1.0 ft of deteriorated concrete. Because the concrete in piers

No. 1, No. 3, No. 4, and No. 5 have been subjected to the same environment, it is assumed that these four piers contain zones of deteriorated concrete similar to the zones in piers No. 6 through No. 10. The additional drilling of piers No. 2 and No. 4 showed that pier No. 2 did have zones of damaged concrete similar to the zones found in piers No. 7 and No. 8.

103. It is recommended that 9 to 12 in. of concrete on exposed surfaces be removed and replaced with new concrete. The exceptions are areas where patched concrete is in good condition and the ceilings in the head gate structure. Localized areas may require deeper removal.

104. The concrete deterioration at Dresden Island Lock and Dam will continue at an accelerated rate due to the exposure in the freezing-and-thawing environment. It is evident from this evaluation that the concrete must be rehabilitated.

References

1. Letter, DAEN-CWE-B (NCCED-DS, 25 June 1973), 2d Ind, subject: Report on Stability Investigation, Dresden Island Lock and Dam, Illinois Waterway; Letter, NCED-T (19 October 1972), 1st Ind, subject: Review and Appropriate Comment - Foundation Report Dresden Island Lock and Dam.
2. Reconnaissance Report, Major Rehabilitation, Illinois Waterway, Dresden Island Lock and Dam, January 1977, Corps of Engineers, Chicago District.
3. "Periodic Inspection, Report No. 2, Dresden Island Lock and Dam. Illinois Waterway," Illinois, Department of the Army, Corps of Engineers, Chicago District, September 1973.
4. "Appendix A, Soils and Geology For Structural Stability Analysis, Dresden Island Lock and Dam, Illinois Waterway," Department of the Army, Corps of Engineers, Chicago District, October 1972, Revised June 1973.
5. Stowe, R.L., "Concrete and Rock Tests, Rehabilitation Work, Brandon Road Dam, Illinois Waterway, Chicago District, to be published as a WES MP.
6. Willman, H., Atherton E., & Others, Handbook of Illinois Stratigraphy Bulletin 95, Illinois State Geological Survey
7. Templeton J., Willman H., Champlanian Series (Middle Odovician) in Illinois Bulletin 89, Illinois State Geological Survey, 1963 p.136
8. Rock Color Chart Committee, National Research Council, Rock Color Chart, Washington, D.C. 1963
9. Letter Report by R. L. Stowe, "Report of Concrete Core Tests, Lock Walls at Dresden Island Lock and Dam, Illinois Waterway, Chicago District," August 1976. On File at Chicago District Office, NCCED-F.
10. Letter Report by R. L. Stowe, "Test Results of Concrete Cores, Dresden Island Lock and Dam, Illinois River, Chicago District," 17 March 1976. On File at Chicago District Office, NCCED-DS.

Table 1

Core Received at WES From Dresden Island Lock and Dam

WES Reference	Drill Hole No.	Date Received	Core Diam. in.	Box No.	Depth, Ft.	For Depth Intervals, Ft.	Top of Hole, Ft.	Remarks
CH-12 DC-1 (A)	GWB1	5-18-77	6	1 of 6	0 - 4.2	497.0-	497.0	Sandy Gravel
CH-12 DC-1 (B)			"	2 of 6	4.2 - 8.0			"
CH-12 DC-1 (C)			"	3	8.0 - 12.0			Clayey Gravel
CH-12 DC-1 (D)			"	4	12.0 - 12.0			"
CH-12 DC-1 (E)			"	5	15.0 - 19.1			"
CH-12 DC-1 (F)	GWB2	5-18-77	"	6	19.1 - 23.4	510.2 - 473.6	510.2	Concrete
CH-12 DC-2 (A)			6	1 of 12	0 - 3.3			Sandy Gravel
CH-12 DC-2 (B)			"	2 of "	3.3 - 6.0			Gravelly Sand
CH-12 DC-2 (C)			"	3 of "	6.0 - 9.0			Sandy Gravel
CH-12 DC-2 (D)			"	4 of "	9.0 - 12.0			"
CH-12 DC-2 (E)			"	5 of "	12.0 - 13.0			"
CH-12 DC-2 (F)			"	6 of "	13.0 - 13.8			Limestone
CH-12 DC-2 (G)			"	7 of "	13.8 - 16.5			"
CH-12 DC-2 (H)			"	8 of "	16.5 - 19.5			"
CH-12 DC-2 (I)			"	9 of "	19.5 - 23.0			"
CH-12 DC-2 (J)			"	10 of "	23.0 - 27.1			"
CH-12 DC-2 (K)			"	11 of "	27.1 - 31.7			"
CH-12 DC-2 (L)			"	12 of "	31.7 - 35.2			Limestone & Shale
CH-12 DC-3	GWB3	5-18-77	6	1 of 1	13.3 - 35.7	499.3 - 463.6 506.8 - 504.0 497.0 -	499.3	Limestone & Shale
CH-12 DC-4		5-18-77	6	1 of 1	0 - 2.8			Shale
CH-12 DC-5 (A)		5-18-77	6	1 of 13	0 - 3.25			Limestone & Shale
CH-12 DC-5 (B)	GW2	5-18-77	"	2 of "	3.25 - 5.65		497.0	Concrete
CH-12 DC-5 (C)			"	3 of "	5.65 - 9.6			"
CH-12 DC-5 (D)			"	4 of "	9.6 - 12.1			"
CH-12 DC-5 (E)			"	5 of "	12.1 - 15.8			"
CH-12 DC-5 (F)			"	6 of "	15.8 - 19.0			"
CH-12 DC-5 (G)			"	7 of "	19.0 - 23.4			"
CH-12 DC-5 (H)			"	8 of "	23.4 - 26.5			"
CH-12 DC-5 (I)			"	9 of "	26.5 - 30.8			Limestone
CH-12 DC-5 (J)			"	10 of "	30.8 - 35.0			Lime., Clay, Shale
CH-12 DC-5 (K)			"	11 of "	35.0 - 39.8			Shale
CH-12 DC-5 (L)			"	12 of "	39.8 - 44.3			"

(Continued)

Table 1 (Continued)

Core Received at WES From Dresden Island Lock and Dam

WES Reference	Drill Hole No.	Date Received	Core Diam. in.	Box No.	Depth, ft.	For Depth Intervals, Ft.	Top of Hole, Ft.	Remarks
CHI-12 DC-1 (M)	GW2	5-18-77	"	13 of 13	44.3 - 48.3	-473.6	489.8	Clay, Shale, lime.
CHI-12 DC-6	GW3	6-14-77	"	1 of 1	0 - 3.0	489.8 - 486.8	486.3	Concrete
CHI-12 DC-7	GW4	6-14-77	"	1 of 1	0 - 3.2	486.3 - 483.1	486.6	Concrete
CHI-12 DC-8	GW4A	6-14-77	"	1 of 1	0 - 3.1	486.6 - 483.5	497.0	Concrete
CHI-12 DC-9 (A)	GW5	5-18-77	"	1 of 13	0 - 4.5	497.0 -		"
CHI-12 DC-9 (B)			"	2 of "	4.5 - 8.2			"
CHI-12 DC-9 (C)			"	3 of "	8.2 - 12.6			"
CHI-12 DC-9 (D)			"	4 of "	12.6 - 16.8			"
CHI-12 DC-9 (E)			"	5 of "	16.8 - 20.5			"
CHI-12 DC-9 (F)			"	6 of "	20.5 - 24.3			"
CHI-12 DC-9 (G)			"	7 of "	24.3 - 26.8			Shale
CHI-12 DC-9 (H)			"	8 of "	26.8 - 31.1			"
CHI-12 DC-9 (I)			"	9 of "	31.1 - 35.4			"
CHI-12 DC-9 (J)			"	10 of "	35.4 - 38.6			"
CHI-12 DC-9 (K)			"	11 of "	38.6 - 42.8			"
CHI-12 DC-9 (L)			"	12 of "	42.8 - 47.0			"
CHI-12 DC-9 (M)			"	13 of "	47.0 - 48.0			"
CHI-12 DC-10	GW5A	5-18-77	6	1 of 1	0 - 2.3	449.0	497.0	Concrete
CHI-12 DC-11	GW5B	5-18-77	6	1 of 1	0 - 3.6	497.0 - 493.4	497.0	Concrete
CHI-12 DC-12	GW6	6-14-77	6	1 of 1	0 - 3.5	489.8 - 486.3	489.8	"
CHI-12 DC-13	GW7	6-14-77	6	1 of 1	0 - 3.1	486.1 - 483.0	486.1	"
CHI-12 DC-14 (A)	GW8	6-14-77	6	1 of 4	0 - 3.3	497.0 -	497.0	"
CHI-12 DC-14 (B)			"	2 of 4	3.3 - 7.3			"
CHI-12 DC-14 (C)			"	3 of 4	7.3 - 19.2			NX Core at 487.7
CHI-12 DC-14 (D)			2.125	4 of 4	19.2 - 33.5			Con. Shale, lime.
CHI-12 DC-15	L1	6-14-77	6	1 of 1	0 - 3.1	463.5	497.0	Concrete
CHI-12 DC-16	L2		"	1 of 1	0 - 3.0	507.5 - 504.4	507.5	Concrete
CHI-12 DC-17	L3		"	1 of 1	0 - 1.9	507.8 - 504.8	507.8	Concrete
CHI-12 DC-18	L4		6	1 of 1	0 - 2.0	500.5 - 498.6	500.5	Concrete
						502.0 - 500.0	502.0	Concrete

(Continued)

Table 1 (Continued)

Core Received at WES From Dresden Island Lock and Dam

WES Reference	Drill Hole No.	Date Received	Core Diam., in.	Box No.	Depth, Ft.	For Depth Intervals, Ft.	Top of Hole, Ft.	Remarks
CHI-12 DC-19	L5	6-14-77	6	1 of 1	0 -2.0	504.3 -506.3	506.3	Concrete
CHI-12 DC-20	L6		"	1 of 1	0 -2.8	489.3 -486.5	489.3	"
CHI-12 DC-21	L7		"	1 of 1	0 -2.9	488.5 -485.6	488.5	"
CHI-12 DC-22	D1	6-14-77	6	1 of 1	0 -2.7	507.7 -505.0	507.7	"
CHI-12 DC-23	D2		"	1 of 1	0 -3.3	488.1 -484.8	488.1	"
CHI-12 DC-24	D3		"	1 of 1	0 -3.3	492.0 -488.7	492.0	"
CHI-12 DC-25	D4		"	1 of 1	0 -3.1	507.4 -504.3	507.4	"
CHI-12 DC-26	D5		"	1 of 1	0 -3.2	507.3 -504.1	507.3	"
CHI-12 DC-27	D6		"	1 of 1	0 -3.1	506.5 -503.4	506.5	"
CHI-12 DC-28	D7		"	1 of 1	0 -3.0	505.5 -502.5	505.5	"
CHI-12 DC-29	D8		"	1 of 1	0 -3.2	489.9 -486.7	489.9	"
CHI-12 DC-30	D10		"	1 of 1	0 -2.9	506.5 -503.6	506.5	"
CHI-12 DC-31	D11		"	1 of 1	0 -3.0	506.4 -503.4	506.4	"
CHI-12 DC-32	D12		"	1 of 1	0 -3.1	490.0 -486.9	490.0	"
CHI-12 DC-33	D13		"	1 of 1	0 -5.2	510.6 -505.4	510.6	"
CHI-12 DC-34	D14		"	1 of 1	0 -3.3	493.0 -489.7	493.0	"
CHI-12 DC-35	D15		"	1 of 1	0 -5.3	489.0 -483.7	489.0	"
CHI-12 DC-36	D16		"	1 of 1	0 -3.2	506.5 -503.3	506.5	"
CHI-12 DC-37	D17		"	1 of 1	0 -5.1	489.0 -483.9	489.0	"
CHI-12 DC-38	D18		"	1 of 1	0 -3.2	492.0 -488.8	492.0	"
CHI-12 DC-39	D19		"	1 of 1	0 -3.1	505.5 -502.4	505.5	"
CHI-12 DC-40	D20		"	1 of 1	0 -3.1	506.8 -503.7	506.8	"

Table 2

Test Results of Limestone Cores, Dresden Island Lock and Dam

Drill Hole No. DI-WES-77	Characterization Tests					Eng. Design Tests	
	Elev., ft.	Effective unit wt., γ_m , lb/ft ³	Dry unit wt., γ_d , lb/ft ³	Water Content, w, %	Comp. Wave Velocity, Vp, fps	Comp. Strength, UC, psi	Elastic Modulus Ex10 ⁶ psi Poisson's Ratio
GW-2	471.1	177.9	176.1	1.0	20,200	15,170	6.82 0.21
GW-8	472.4	176.7	175.6	0.6	18,000	7990	5.17 0.34
GW-8	471.3	175.4	174.0	0.8	16,315	9200	4.29 0.20
GW-8	469.4	176.1	174.2	1.1	17,500	8980	5.77 0.28
Average		176.5	175.0	0.9	18,004	10,340	5.51 0.26
Range		2.5	2.1	0.5	3885	7,180	2.53 0.14
No. of Tests		4	4	4	4	4	4

Table 3

Test Results of Shale Cores, Dresden Island Lock and Dam

Characterization Tests					Tensile Splitting Strength, Ts, psi	Eng. Design Tests		
Drill Hole No. DI-WES-77	Elev. ft.	Effective unit wt, γ_m , lb/ft ³	Dry unit wt, γ_d , lb/ft ³	Water Content, w, %		Comp. Strength- UC, psi	Elastic Modulus, Ex10 ⁶ , psi	Poisson's Ratio
GW-2	462.5	152.3	143.1	6.4	8783	1230	0.25	0.37
GW-2	460.4	154.8	142.7	8.5	8305	950	0.24	0.40
GW-2	457.7	159.2	148.5	7.2	9245	1520	0.25	0.30
GW-5	460.1			5.3		225		
GW-5	456.5			6.0		175		
GW-5	453.5			7.8		185		
Average		155.4	144.8	6.9	8778	1230	0.25	0.36
Range		6.9	5.8	3.2	940	570	0.01	0.10
No. of Tests		3	3	6	3	3	3	3

Table 4

Laboratory Test Results - Dresden Island Lock and Dam

Single Plane Shear Tests

Lithology	Drill Hole No. DI-WES- -77	Elev., ft.	Test	Normal Stress, tsf	Peak Shear Stress, tsf	Ultimate Shear Stress, tsf	Peak Shear Strength, psi	Ultimate Shear Strength, psi
Limestone	GW-2	473.3	Concrete to Rock	2	16.70	5.10	c=19.05	c=0.0
	GW-2	472.0		4	39.20	3.50	$\phi=67.2^\circ$	$\phi=37^\circ$
	GW-2	471.8		8	34.50	9.00		
Shale	GW-5	471.0	Concrete to Rock	2	6.90	6.90	c=5.56	c=0.0
	GW-5	469.6		4	11.60	11.60	$\phi=49^\circ$	$\phi=49^\circ$
	GW-5	466.3		8	14.30	14.30		
Shale	GW-2	465.4	Intact	2	4.11	3.84	c=3.56	c=0.0
	GW-2	464.9		4	10.30	10.30	$\phi=47.2^\circ$	$\phi=42.0^\circ$
	GW-2	459.5		8	11.40	10.20		
Shale	GW-2	461.6	Cross Bed, 45°	2	7.90	7.90	c=5.65	c=0.0
	GW-2	456.5		4	9.60	9.60	$\phi=46.3^\circ$	$\phi=46.3^\circ$
	GW-2	453.0		8	14.10	14.10		
Shale	GW-5	466.1	Precut	2	1.10	1.02	c=0.22	$\phi=0.0$
				4	1.64	1.60	$\phi=21.4^\circ$	$\phi=20.2^\circ$
				8	3.40	3.20		
Shale	GW-5	464.7	Precut	2	2.50	2.20	c=1.2	c=0.0
				4	2.40	2.30	$\phi=24.2^\circ$	$\phi=24.2^\circ$
				8	5.00	5.00		
Dolomitic Shale	GW-5	463.1	Precut	2	1.50	1.50	c=0.1	c=0.0
				4	1.60	1.60	$\phi=27.2^\circ$	$\phi=27.2^\circ$
				8	4.40	4.40		

Table 5

Concrete Tests Results, Dresden Island Lock and Dam

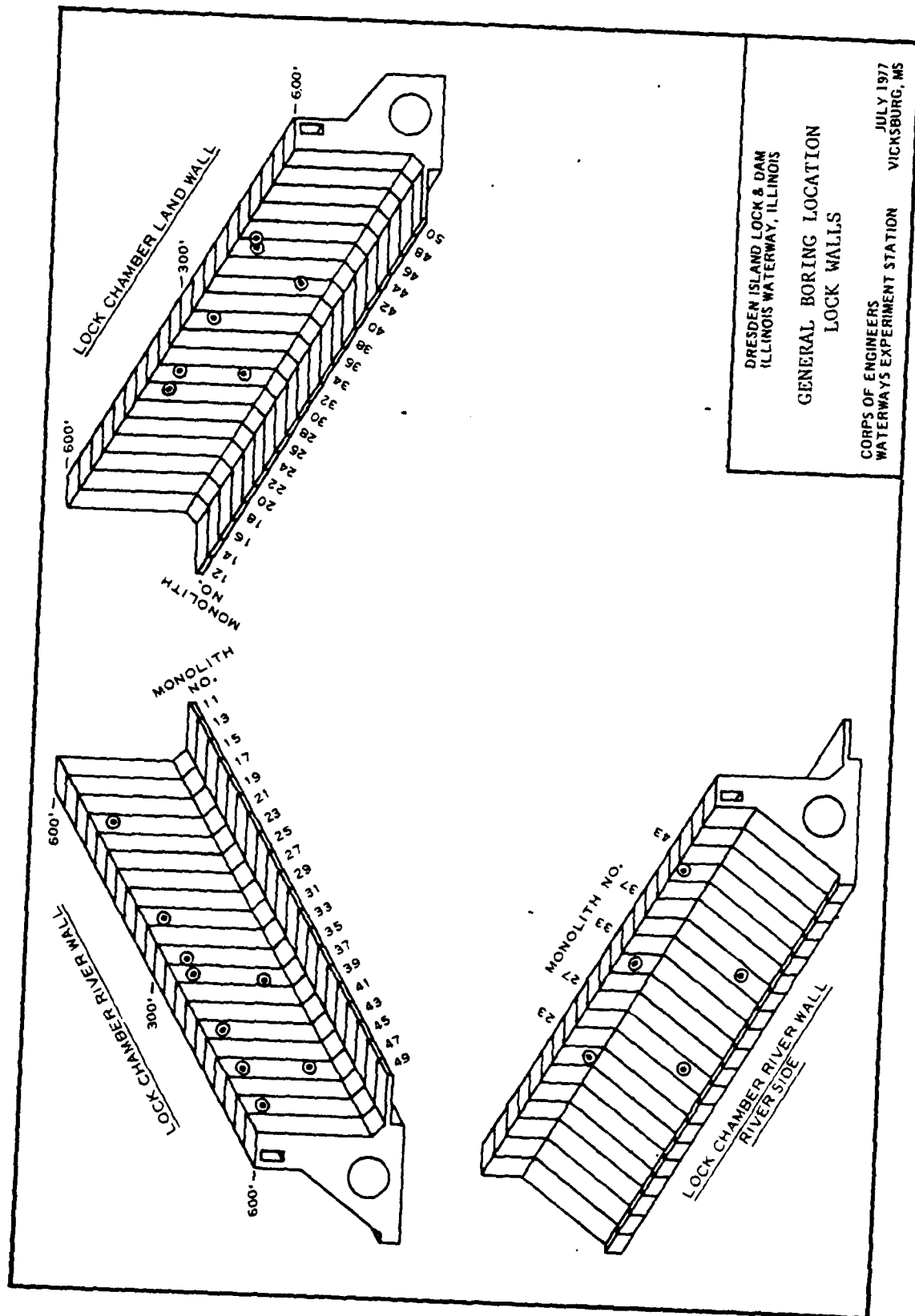
Drill Hole No. DI-WES-77	Depth of Core, ft.	Elev., ft.	Characterization Tests				Comp. Strength, UC, psi	Eng. Design Tests Elastic Modulus- Ex10 ⁶ , psi, Ratio
			Effective unit wt, γ_m , lb/ft ³	Dry unit wt, γ_d , lb/ft ³	Water Content, w, %	Comp. Wave Velocity, Vp, fps		
GW-1	2.3	504.5	147.3	139.5	5.6	14,764	7840	5.33
GW-2		493.3	148.0	135.7	9.1	14,750	3810	4.05
GW-2		483.5	149.2	138.1	8.0	16,350	5050	4.17
GW-2		475.0	151.1	139.9	8.0	16,835	6120	5.26
GW-3	2.5	487.3	149.2	142.0	5.1	13,698	3880	4.55
GW-4A	0.5	486.1	145.5	135.6	7.3	12,500	4870	2.76
GW-5		493.8	147.3	137.7	7.0	14,990	4520	5.00
GW-5		483.8	146.7	136.7	7.3	14,925	5490	4.65
GW-5		473.9	148.0	138.1	7.2	15,150	4560	4.44
GW-7	1.7	484.4	148.6	136.6	8.8	15,015	4850	4.21
L-2	2.5	503.3	148.0	142.4	8.5	13,861	4460	3.81
L-3	1.3	499.2	150.5	142.4	5.7	14,285	5610	3.20
L-4	1.5	500.5	149.2	140.0	6.6	15,149	4190	5.00
L-5	0.5	505.8	146.7	137.1	7.0	14,333	2520	2.35
L-5	1.5	504.8	146.7	138.7	5.8	15,750	5290	4.44
L-6	0.5	488.8	146.7	137.1	7.0	13,333	5580	2.90
L-6	2.3	487.0	147.3	137.4	7.2	14,142	6180	4.08
L-7	1.1	487.4	147.3	137.3	7.3	14,084	5760	3.40
L-7	2.4	486.1	149.8	139.3	7.5	14,171	5220	4.00
D-2	0.5	487.6	147.3	136.6	7.8	12,666	2930	2.35
D-2	2.8	485.3	146.1	137.4	6.3	13,131	6080	4.26
D-4	0.9	506.5	147.3	137.2	7.4	13,486	4610	3.14
D-4	2.6	504.8	147.3	138.1	6.7	14,400	5360	4.44
D-5	2.6	504.7	148.6	139.3	6.7	14,285	4600	4.85
D-6	2.1	504.4	148.6	137.5	8.1	15,531	4440	4.08
D-7	0.5	505.0	148.6	139.7	6.4	15,593	4940	4.76
D-7	2.2	503.3	150.5	142.8	5.4	15,936	4690	4.85

(Continued)

Table 5

(Continued)

Drill Hole No. DI-WES-77	Depth of Core ft.	Characterization Tests					Eng. Design Tests	
		Elev. ft.	Effective unit wt., γ_m , lb/ft ³	Dry unit wt., γ_d , lb/ft ³	Water Content, w, %	Comp. Wave Velocity, V _p , fps	Comp. Strength, UC, psi	Elastic Modulus, Ex10 ⁶ , psi
D-8		487.8	148.0	138.2	7.1	14,705	3610	4.00
D-11	2.1	504.9	148.6	139.1	6.8	14,000	4870	2.96
D-11	1.5	503.9	149.2	139.0	7.3	14,830	4720	4.00
D-13	2.5	509.4	146.7	137.0	7.1	13,513	4350	2.62
D-13		506.1	146.7	137.2	6.9	13,698	5130	3.88
D-14		490.2	146.1	136.9	6.7	15,046	4770	3.70
D-15	2.8	484.2	149.2	140.5	6.2	15,936	5140	4.71
D-16		505.4	149.2	137.9	8.2	12,550	4690	1.86
D-16	1.1	503.8	144.8	132.6	9.2	14,084	5240	3.92
D-18	2.7	491.1	145.5	135.7	7.2	14,764	4740	3.20
D-18	0.9	489.3	146.7	136.1	7.8	16,096	4840	5.00
D-18	2.7							0.28
Average		147.8	148.0	138.0	7.1	14,535		
Range		6.3	10.2	4.1	4.1	4335		
No. of Tests		38	38	38	38	38		



DRESDEN ISLAND LOCK & DAM
ILLINOIS WATERWAY, ILLINOIS

GENERAL BORING LOCATION
LOCK WALLS

CORPS OF ENGINEERS
WATERWAYS EXPERIMENT STATION

JULY 1977
VICKSBURG, MS

Plate 1

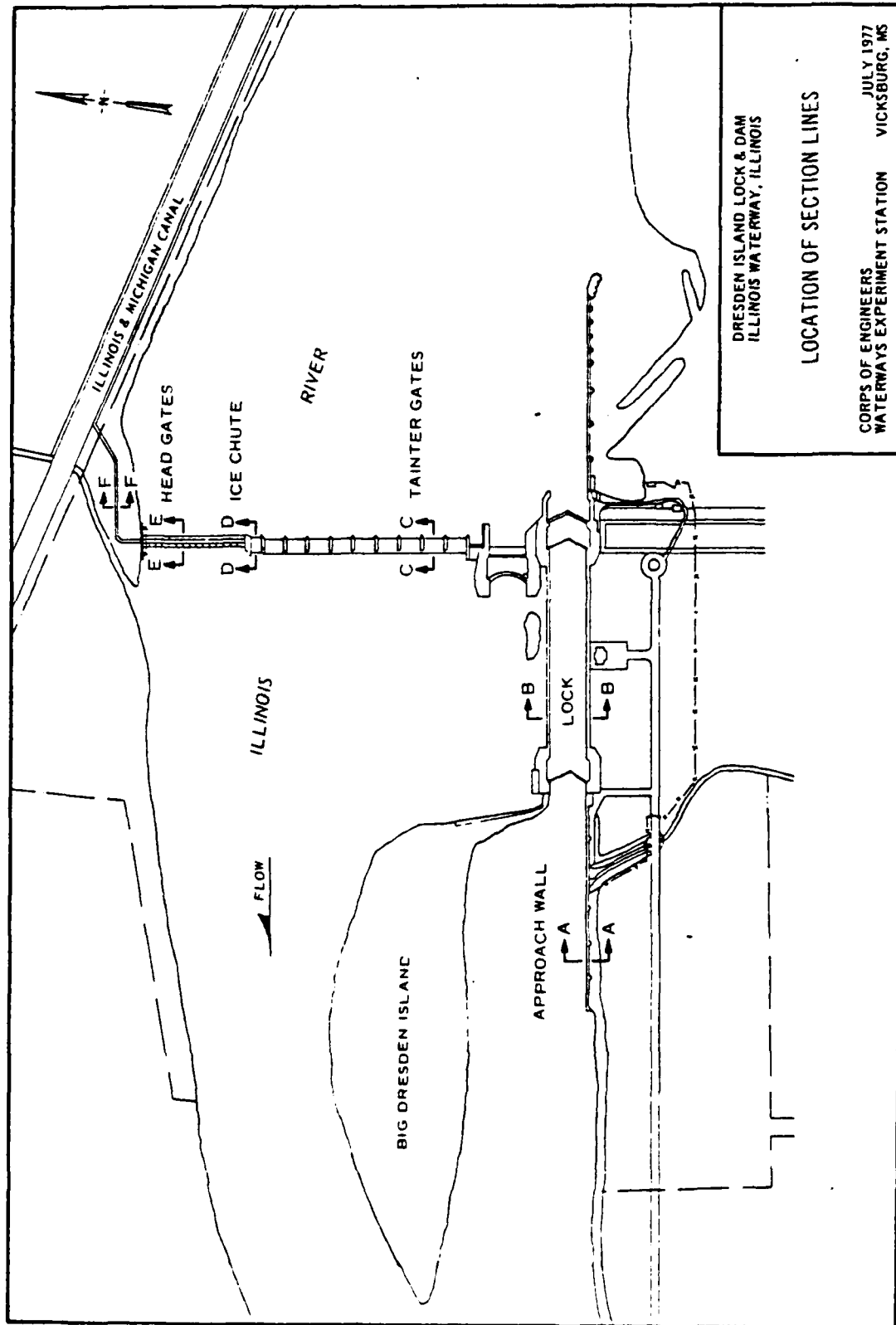
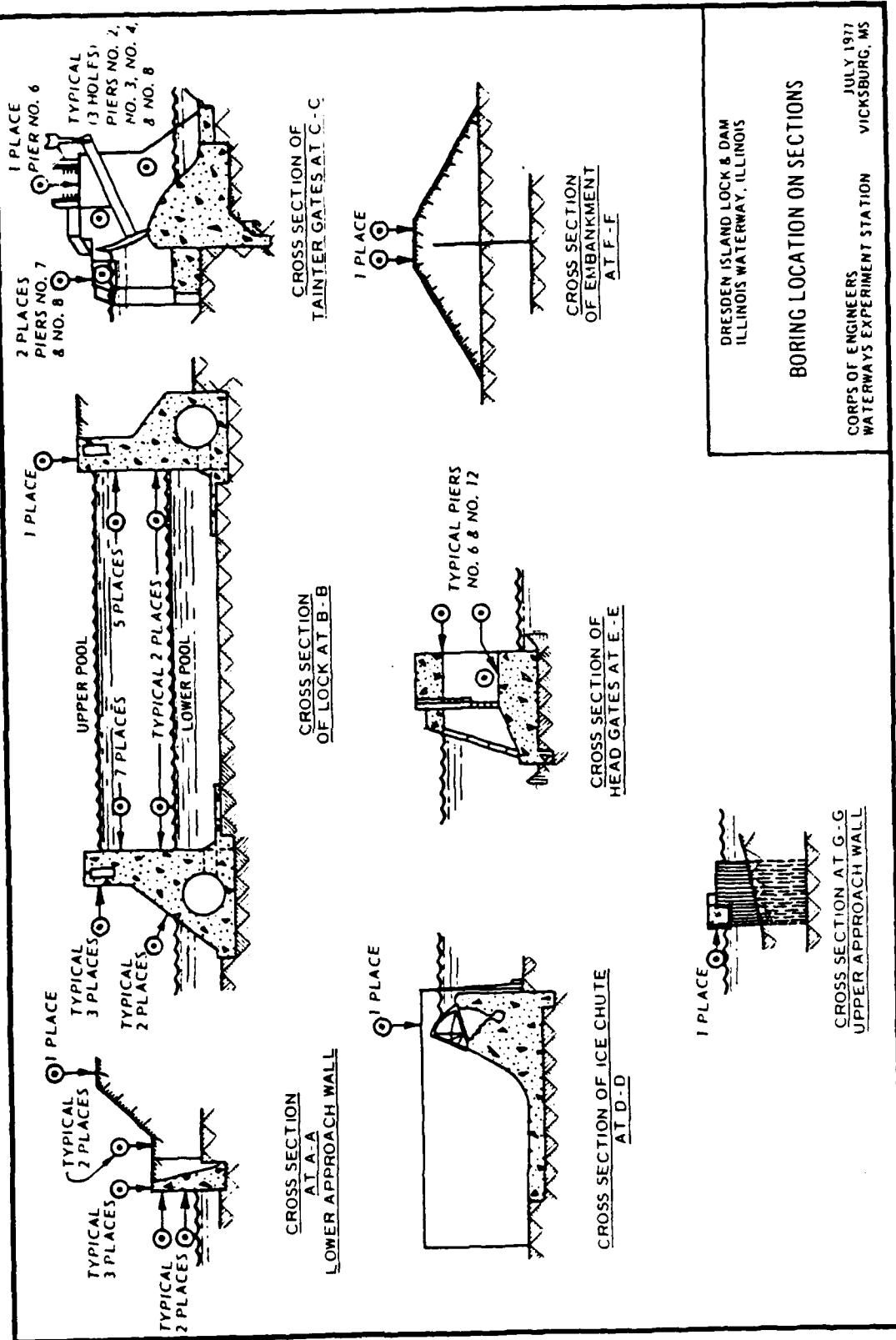


Plate 2a



PIEZOMETER INSTALLATION REPORT

PROJECT: Dresden Island Lock						LEVEE DISTRICT: Chicago District					
LOCATION (STA): See Note (1)				OFFSET FROM CENTER LINE: -				PIEZ NO.: -			
SILICA SAND BONDED TO PIEZ TYPE: Slotted, 1 1/2" PVC Pipe (2)				DEPTH OF PIEZ: 19.1'				RISER PIPE DIAM: 1 1/2"			
PIEZ TIP SET IN (SOIL TYPE): Rip-Rap Backfill				SOIL SAMPLE NO.: -				BORING DIAM: 5 1/2"			
METHOD OF INSTALLATION: Piezometer set in open hole.											
TYPE OF PROTECTION: Riser encased in concrete FOR PIEZ from Elev. 483.5 to the ground surface.								VENT: 1/8" dia. hole in cap			
GROUND ELEV: 497.0				ELEV TOP OF RISER: 498.9				ELEV PIEZ TIP: 477.9 (3)			
FILTER: Concrete Sand				FROM ELEV: 473.6				TO ELEV: 482.9			
SEAL: Bentonite Balls				FROM ELEV: 482.9				TO ELEV: 483.5			
INSTALLED BY: WES						CONTRACT NO.:			FOREMAN: McSee		
DATE OF INSTALLATION: 23 Apr. 1977						DATE OF OBSERVATIONS: (4)					
METHOD OF TESTING PIEZ: (4)											
TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET
REMARKS (1) Piez. set in Boring DI-WES-SWB-1-77											
(2) C.D. of bonded silica tip is 3.0", length is 1.0'.											
(3) Elevation of tip bottom.											
(4) No observations or tests run on piez. as of 5 May 1977											

WES FORM 798
MAR 53
REVISED OCT 53

INSPECTOR

Plate 3

PIEZOMETER INSTALLATION REPORT











PROJECT: Dresden Island Lock						LEVEE DISTRICT: Chicago District					
LOCATION (STA): See Note (1)				OFFSET FROM CENTER LINE: -				PIEZ NO.: -			
PIEZ TYPE: Silica Sand Bonded to Slotted, 1 1/2" PVC Pipe (2)				DEPTH OF PIEZ: 31.8				RISER PIPE DIAM: 1 1/2"			
(SOIL TYPE): Rip-Rap Backfill				SOIL SAMPLE NO.: -				BORING DIAM: 7 3/4"			
METHOD OF INSTALLATION: Piezometer set in open hole.											
TYPE OF PROTECTION Riser encased in concrete FOR PIEZ from Elev. 501.2 to the ground surface						VENT: 1/8" dia. hole in cap.					
GROUND ELEV: 510.2 (3)				ELEV TOP OF RISER: 510.4				ELEV PIEZ TIP: 478.4 (4)			
FILTER: Concrete Sand FROM ELEV: 475.0						TO ELEV: 500.2					
SEAL: Bentonite Balls FROM ELEV: 500.2						TO ELEV: 501.2					
INSTALLED BY: WES						CONTRACT NO.:				FOREMAN: McGee	
DATE OF INSTALLATION: 27 Apr. 1977						DATE OF OBSERVATIONS: (5)					
METHOD OF TESTING PIEZ: (5)											
TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET
REMARKS: (1) Piez. set in Boring DI-WES-GWB-2-77											
(2) O.D. of bonded silica tip is 3.0", length is 1.0'.											
(3) Elevation determined with a hand level.											
(4) Elevation of tip bottom.											
(5) No observations or tests run on piez. as of 5 May 1977.											

WES FORM 798
MAR 53
REVISED OCT 53

INSPECTOR

Plate 4

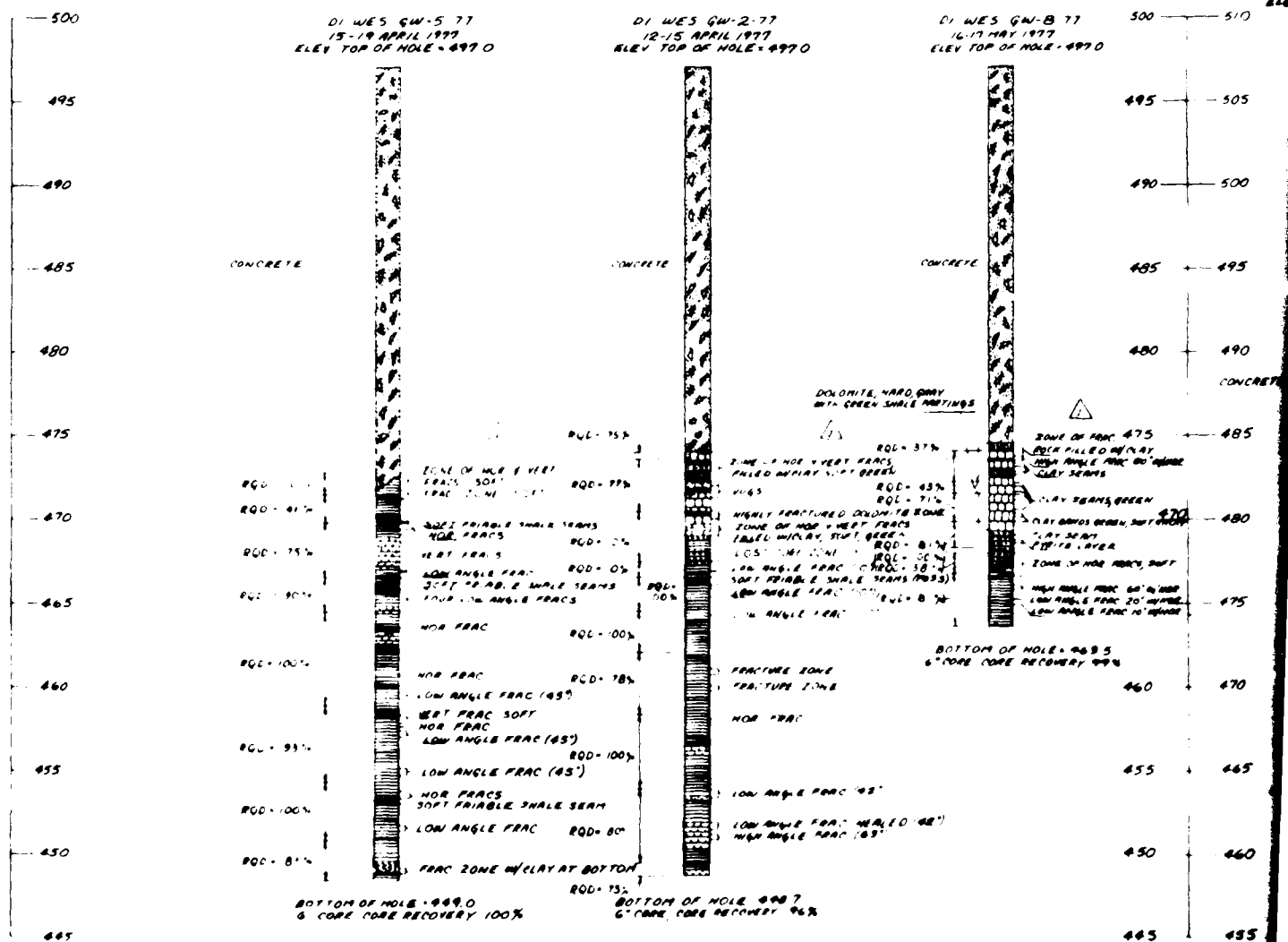
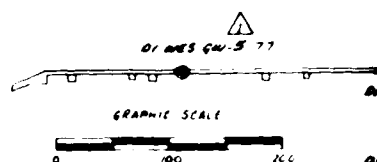
SYMBOLS

	LIMESTONE		CLAYEY SHALE
	SHALE		FOSSIL LAYER
	CLAY		CALCAREOUS SHALE
	SHALY CLAY		CONCRETE
	SALTSTONE		DOLOMITIC SHALE

REMARKS

BORINGS THIS SHEET ARE BY THE W & S DRILL
CREW. GEOLOGIC DESCRIPTIONS ARE BY W & S
ALSO.

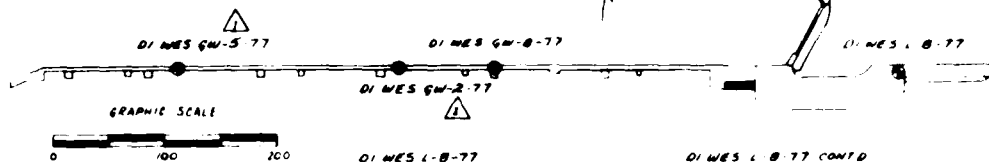
BORING LOCATION PLAN



REMARK 5

FORINGS THIS SHEET ARE BY THE W & S DRILL
TEAM. GEOLOGIC DESCRIPTIONS ARE BY W & S
AS SO

BORING LOCATION PLAN

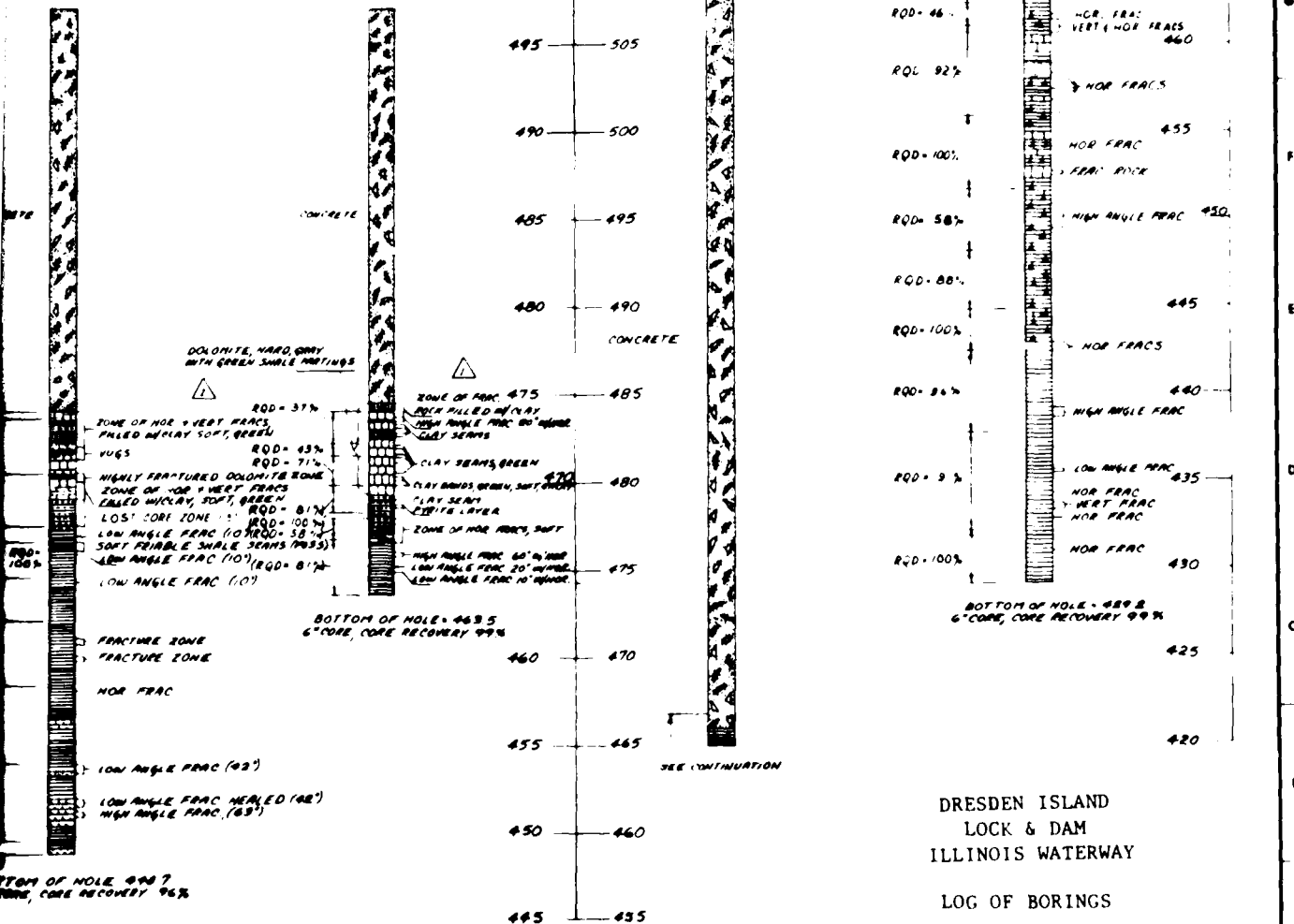


WES GW-2 77
12-15 APRIL 1977
R. TOP OF HOLE: 4970

DI WES GW-8-77
17 MAY 1977
ELEV TOP OF HOLE - 4970

01 WES L-B-77
20-25 AUG. 1977
ELEV. TOP OF HOLE 510.0

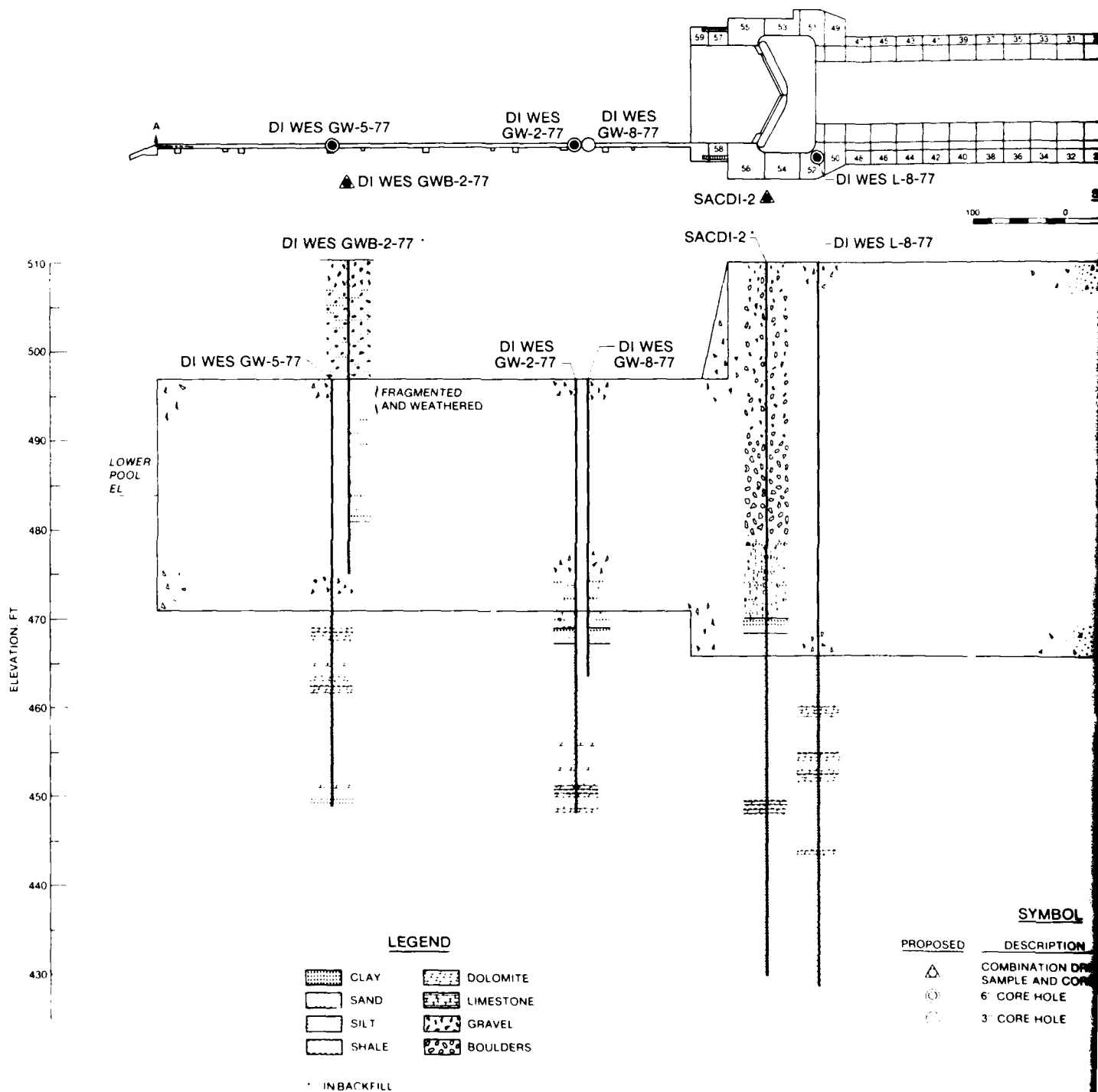
DI WES 4-8-77 CONT'D

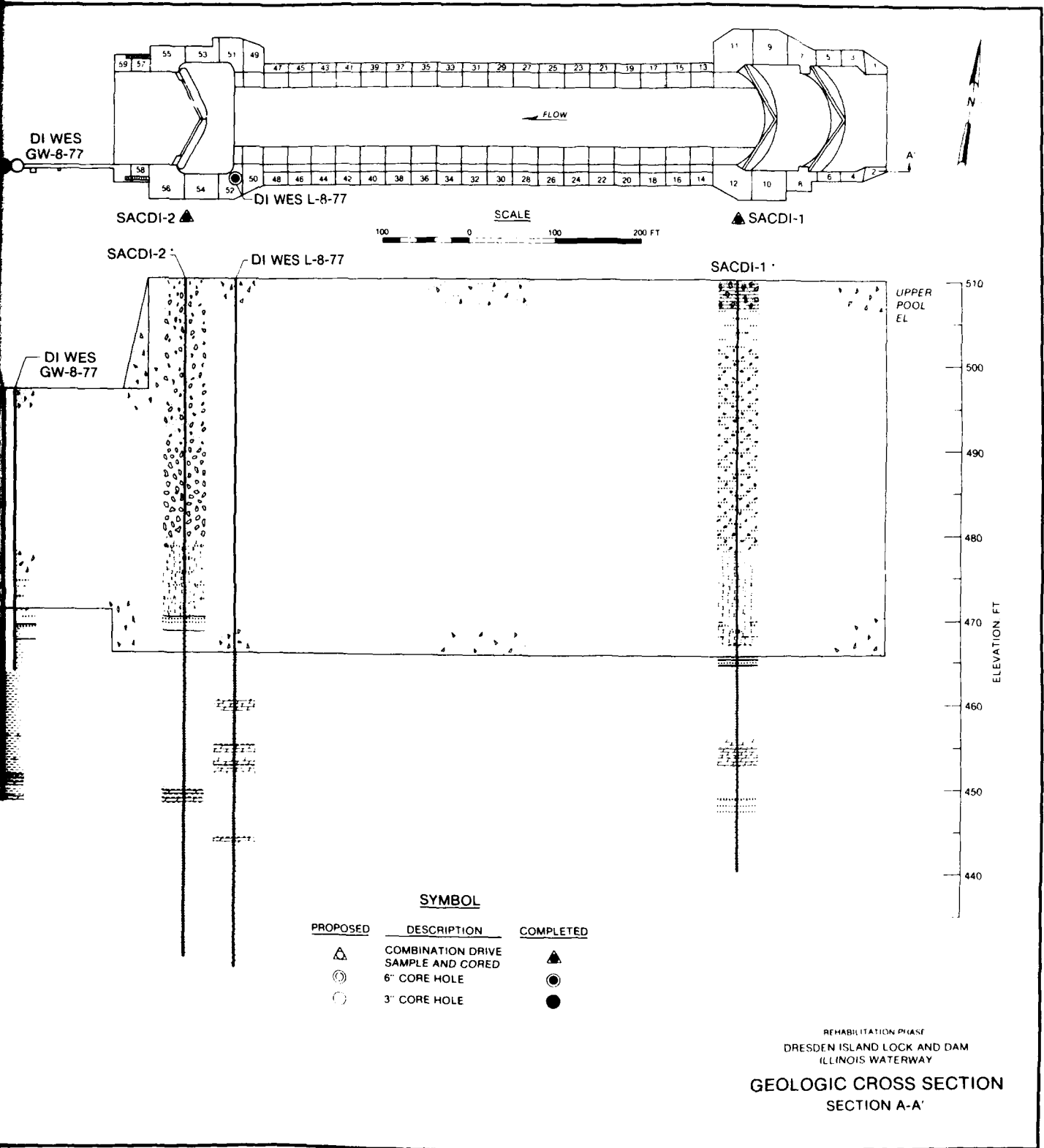


DRESDEN ISLAND
LOCK & DAM
ILLINOIS WATERWAY

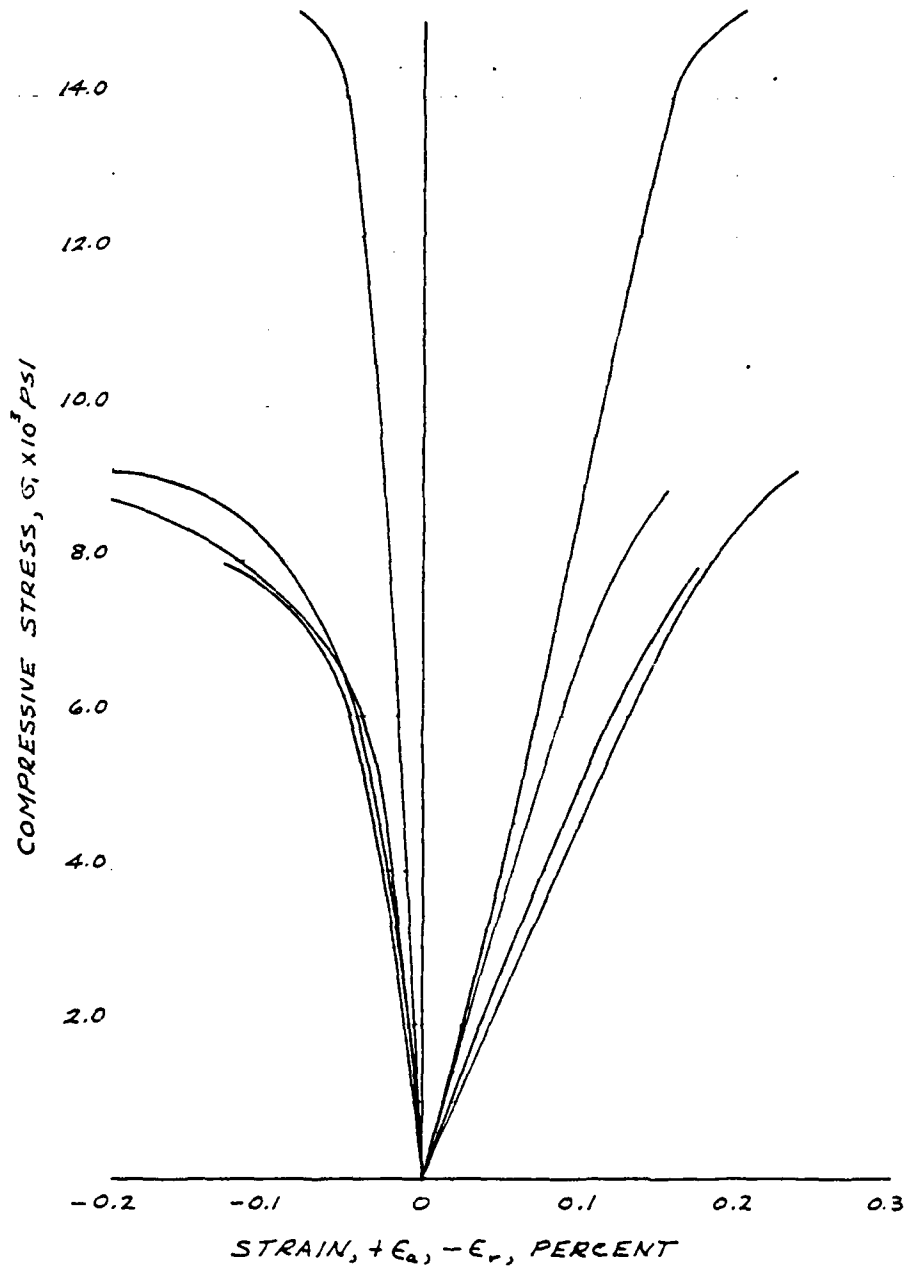
LOG OF BORINGS

DATE MAY 1978



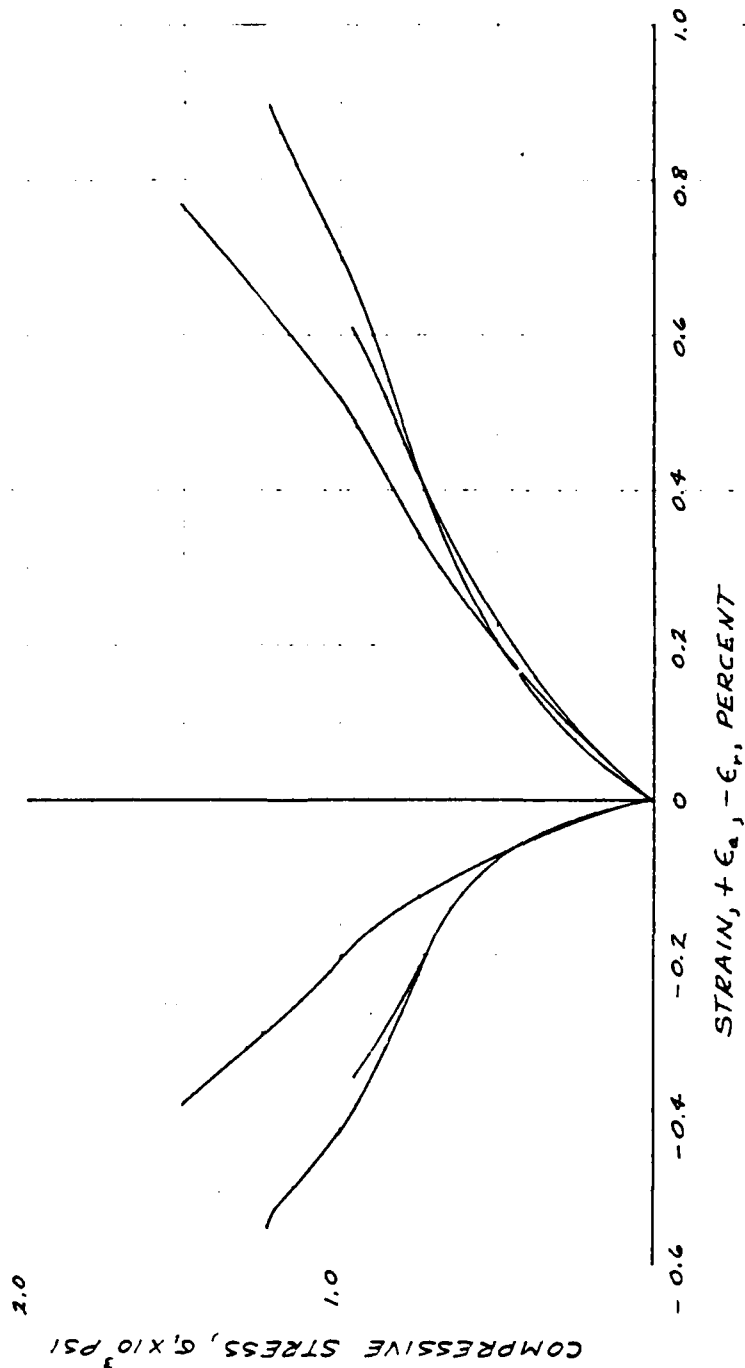


Hole No.	Elev. Ft.	σ_1 PSI	Modulus $E \times 10^6$ PSI	Poisson's Ratio	Water Content Percent
GW-2	471.1	15,170	6.82	0.21	1.0
GW-8	472.4	7990	5.17	0.34	0.6
GW-8	471.3	9200	4.29	0.20	0.8
GW-8	469.4	8980	5.77	0.28	1.1



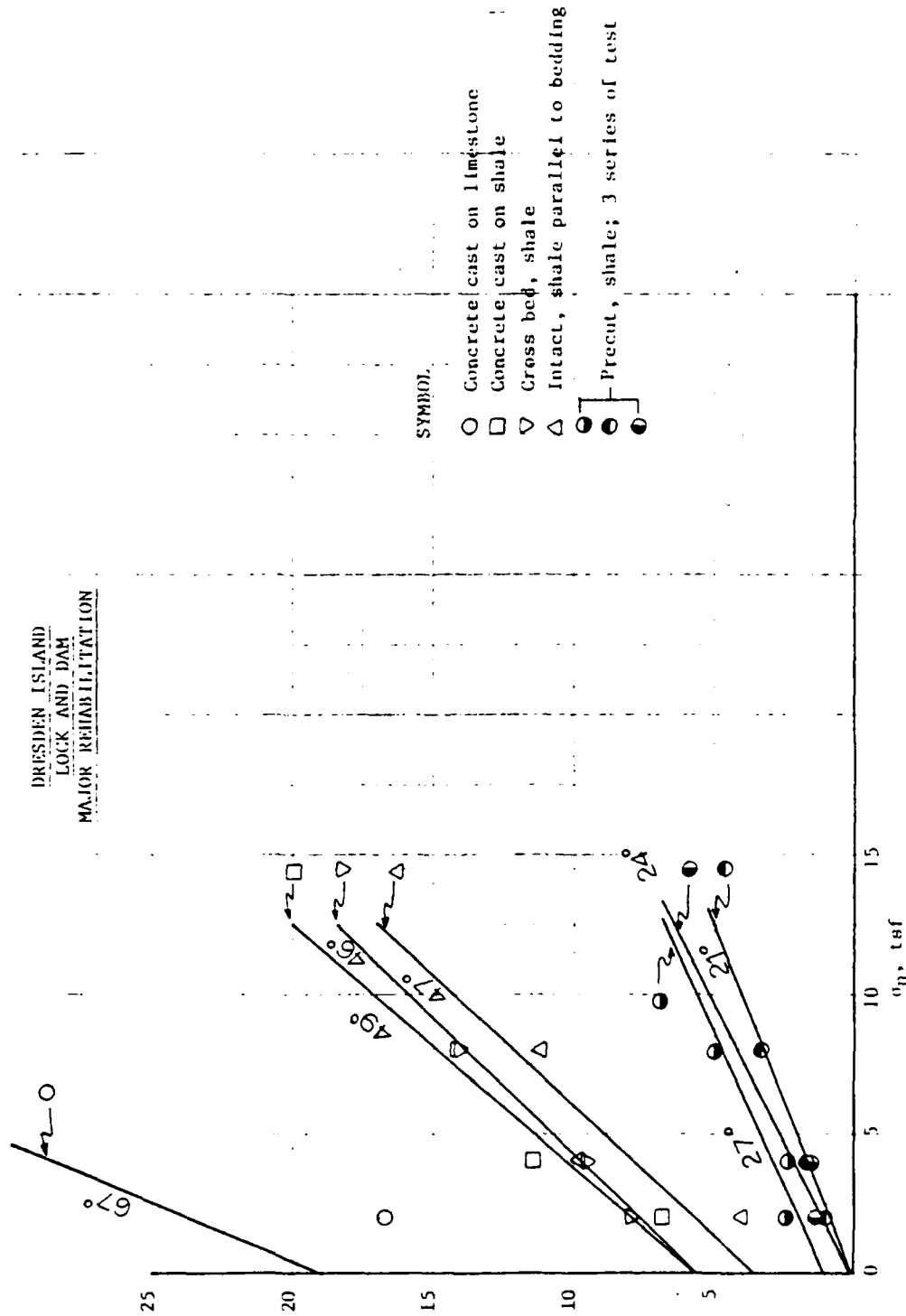
STRESS-STRAIN RELATION FOR LIMESTONE, DI LOCK DAM

Hole No.	Elev Ft.	σ_1 PSI	Modulus $E \times 10^6$ PSI	Poisson's Ratio	Water Content Percent
GW-2	462.5	1230	0.25	0.37	6.4
GW-2	460.4	950	0.24	0.40	8.5
GW-2	457.7	1520	0.25	0.30	7.2



STRESS-STRAIN RELATION FOR SHALE, DI LOCK DAM

DRESDEN ISLAND
LOCK AND DAM
MAJOR REHABILITATION



Project: Dresden Inland Lock and Dam		General Description of Concrete in the Guide Wall								COMPUTED BY: _____		DATE: _____
Depth, ft	EL. 506.1 CV-1	EL. 497.0 CV-2	EL. 489.8 CV-3	EL. 486.3 CV-4	EL. 497.0 CV-5, CV-5A, & CV-5B	EL. 489.8 CV-6	EL. 486.1	EL. 497.0 CV-8			DATE: _____	
0.0	Finished Surface Steel Plate 1/2-in. Steel Bar 1-1/4-in. Steel Bar New Concrete	Finished Surface New Concrete Form Surface Contact (old) Old Concrete 3-in. Max Size AGG	Finished Surface 6-in. Max Size AGG	Weathered Scales 2-in. Max Size AGG. Not Carb Form Ties C.	Form Surface New Concrete 3/4-in. Max Size AGG Nat Carb CA Air Entrained	Slightly Spalled 3-in. Max Size AGG FR	3-in. Max Size AGG	Finished Surface 3/4-in. Max Size AGG				
0.5	Contact Concrete above & below contact air entrained			No Entrained Air Steel Cable 1 ft		Old Break Along Length of Core Longitudinal Fracture Filled w/Hebula Transverse Cracks are new	JPR	New Concrete				
1.0	1-in. Max Size AGG. Nat. Carb. CA No Frost Damage Saved	No Frost Damage	No Frost Damage	Max Depth of Frost Damage Not Recovered	Contact Old 3-in. Max Size AGG Nat Carb CA Not Air Entrained		Max Depth of Frost Damage	Contact 6-in. Max Size AGG No Frost Damage				
1.5												
2.0												
2.5												
3.0												

1. Detailed Petrography
2. Holes Continuing Into Foundation Rock
3. Extra Borings Not Reported in This Graphy

1. Detailed Petrography
2. Holes Continue Into Foundation Rock
3. Extra Borings Not Reported in This Graphy

1953A

Subject:		General Description of Concrete in the Lock Wall										Computed by:		Date:	
Breaden Island Lock and Dam		EL.	EL.	EL.	EL.	EL.	EL.	EL.	EL.	EL.	EL.	Checked by:			
Depth, ft	1-1	507.5	507.8	1-2	500.5	502.0	506.3	516.3	489.3	418.5	1-7				
0.0															
0.5															
1.0															
1.5															
2.0															
2.5															
3.0															

* Detailed Petrography Concrete

Project		General Description of Concrete in the Arch Dam and Spillway Dam						Concrete by:		DATE:	
Brecken Inland Lock and Dam		FL	FL	FL	FL	FL	FL	Concrete by:	DATE:	Concrete by:	DATE:
Depth, ft	507.7	498.1	492.0	507.4	507.3						
00	D-3	D-2	D-3	D-4	D-5						
0.5	Finished Surface 1-in. Max Size Agg Crushed 1.5 New Contact	3-in. Max Size Agg Nat Carb CA	Max Depth of Frost Damage Nat Carb CA 3-in. Max Size Agg	FR 3-in. Max Size Agg Nat Carb CA	Finished Surface 3-in. Max Size Agg Nat Carb CA No Air						
1.0	Contact Old Conc No Air 1-1/2-in. Max Size Agg Nat Carb CA Cracking				Max Depth of Frost Damage						
1.5											
2.0											
2.5											
3.0											

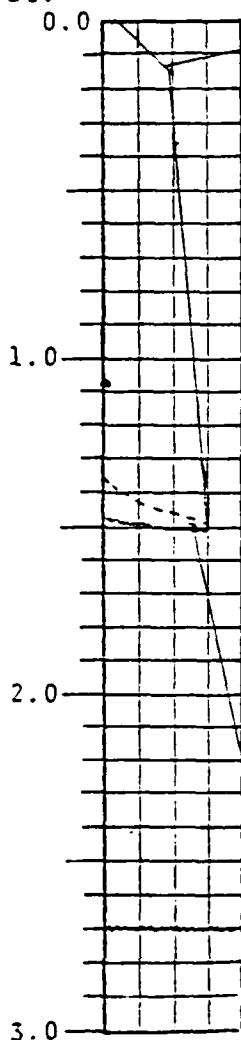
Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152 mm. (6-in.) Diameter Vertical Concrete Core

DI WES GW-5-77

842' Downstream from Center of Lock, 3' Left of Riverside

Depth
ft.

Classification of Materials



Finished Surface

Form Surface

3/4-in. Steel Bar

Possible Frost Damage

Contact Tight But Some Vugs At Contact
 Break Old Calcite Coated

New

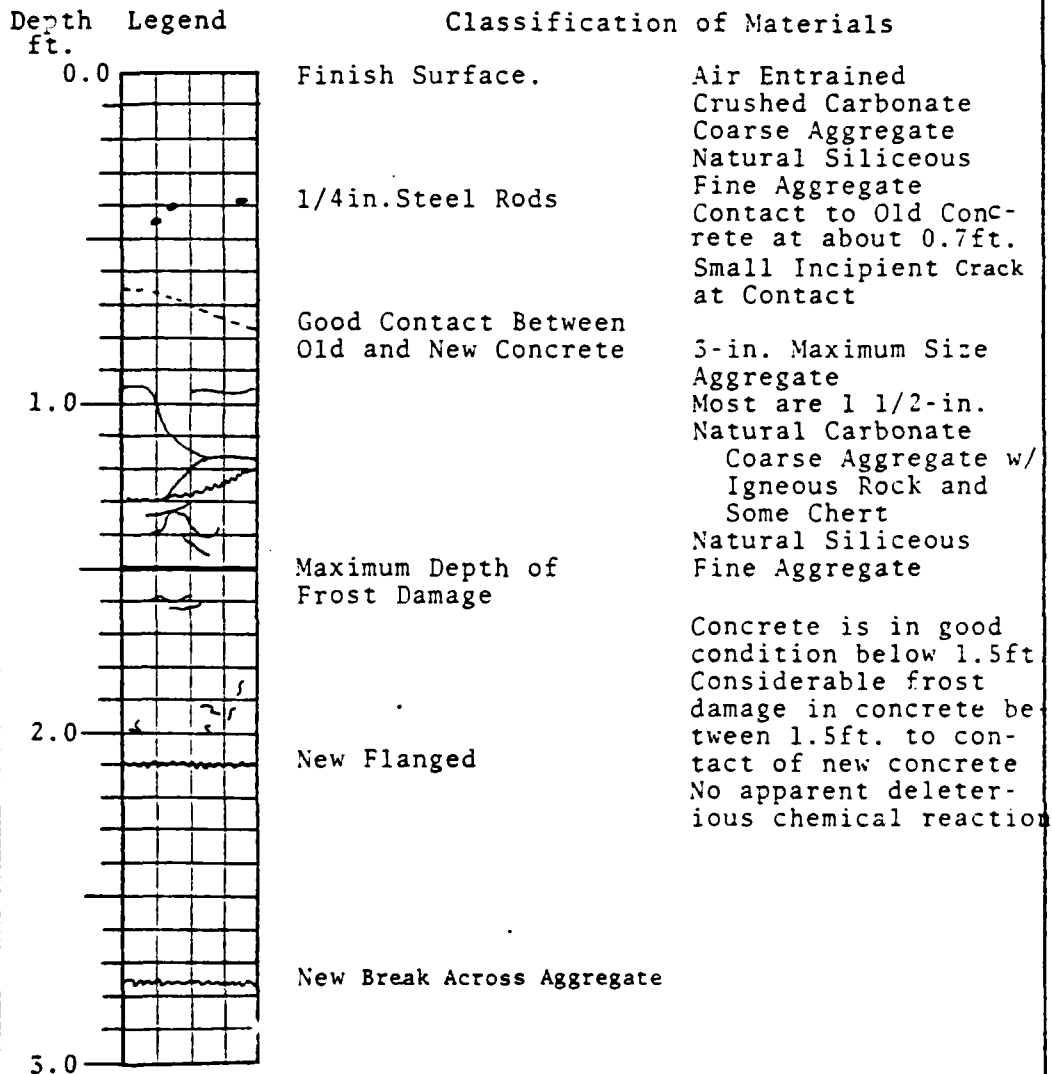
5/4-in. Maximum Size
 Aggregate
 Natural Carbonate
 Coarse Aggregate w/
 some Igneous Rock
 Natural Siliceous
 Fine Aggregate
 Air Entrained
 Some Entrapped Air
 Light Gray paste

Old Concrete Brown-
 Gray Paste
 3-in. Maximum Size
 Aggregate
 Natural Carbonate
 Coarse Aggregate w/
 Some Igneous Rock and
 Chert
 Natural Siliceous
 Fine Aggregate
 Not Air Entrained
 Some Air Entrapped
 Concrete in good con-
 dition except for at
 about 1.5 ft.

Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152 mm. (6-in.) Diameter Horizontal Concrete Core

DI WES D-1-77

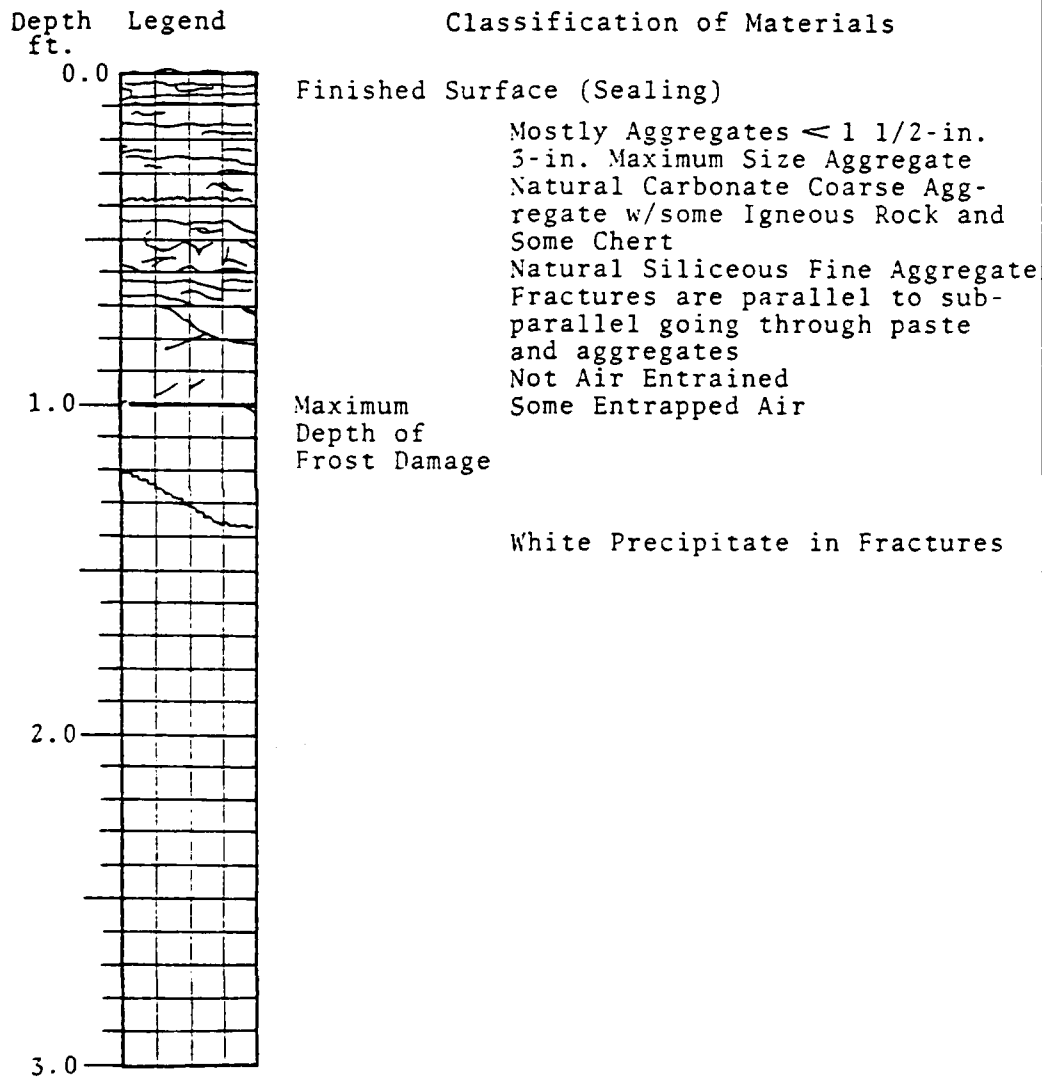
28.7 ft. Upstream of South Corner of Arch Dam Embayment



Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152mm. (g-in.) Diameter Horizontal Concrete Core

DI WES D-5-77

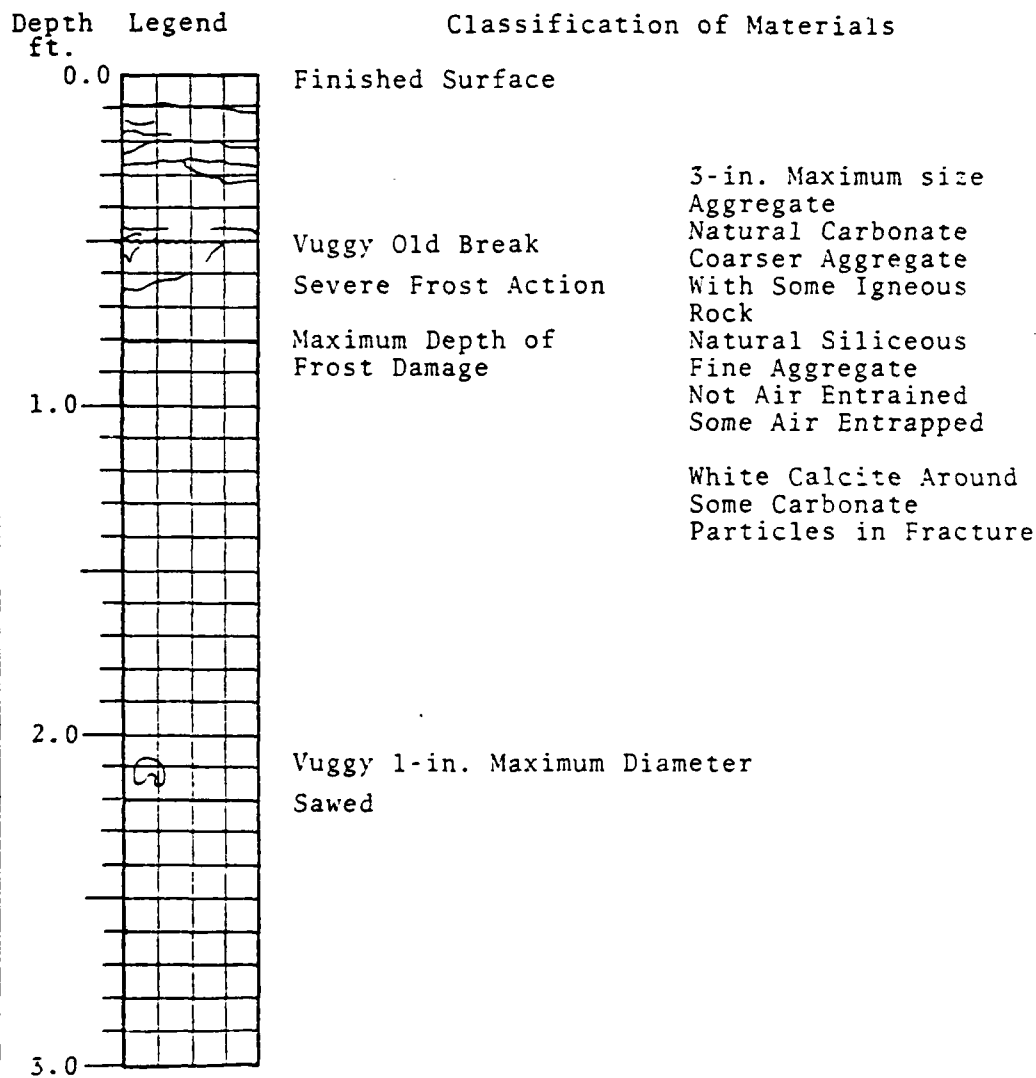
20.8 ft. Upstream from Arch Dam Side of Overflow Weir



Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152mm. (6-in.) Diameter Horizontal Concrete Core

DI WES D-14-77

Pier 6, 8 ft. Upstream from Down Stream Corner,
 4 ft. Up from Floor, North Side of Pier



Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152 mm. (6-in.) Diameter Horizontal Concrete Core

DI WES GW-4-77

585 Downstream from Centerline of Lock, Riverside of Lower Guide Wall

Depth Legend
 ft.

Classification of Materials

0.0

1.0

2.0

3.0

1 1/2-in. Maximum Size Aggregate
 Natural Carbonate Coarse Aggregate
 w/some Igneous Rock
 Natural Siliceous Fine Aggregate
 Not Air Entrained
 Some Air Entrapped

Detailed Examination

Maximum Depth of Frost Damage
 Core Spun

Steel Cable at about 1 ft.

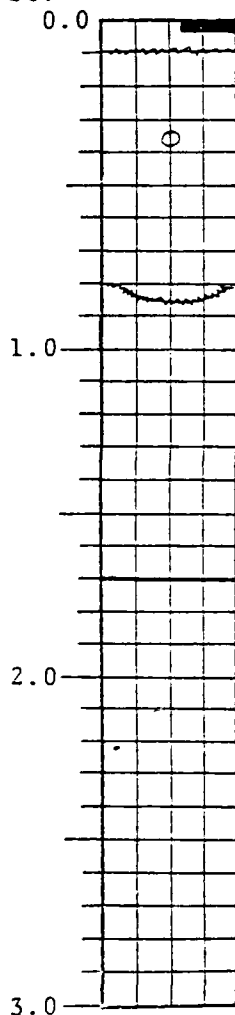
Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152 mm. (6-in) Diamter Horizontal Core

DI WES GW-1-77

774 ft. Upstream From Center Line of Lock, Riverside Guide Wall

Depth Legend
 ft.

Classification of Materials



Finished Surface & 3/4-in. Steel Plate
 3/4-in. Maximum Size Aggregate
 Natural Carbonate Coarse Aggregate w/Some
 Igneous Rock
 Natural Siliceous Fine Aggregate
 1/2-in. Steel Bar
 Air Entrained
 Some Entrapped Air

1/8-in. Steel Fiber
 Contact of Light Brown Concrete and Light
 Gray Concrete
 1-in. Maximum Size Aggregate

Most of the break at 0.8ft. was through
 aggregate and paste but some was along
 interface of two different concrete pours

sawed

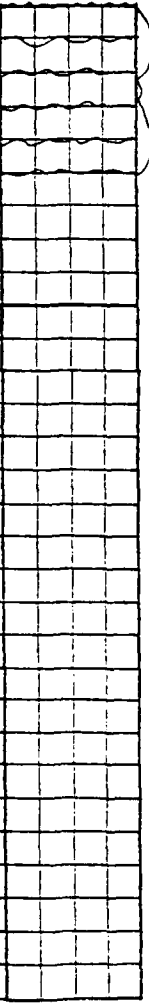
Concrete is in good condition

Dresden Island Lock and Dam
Illinois Waterway, Chicago District
152 mm. (6-in.) Diameter; Core Normal to Slope Surface

DI WES L-7-77

14.8 ft. Upstream from Center Line of Lock,
Lock Chamber River Wall, Riverside

Depth ft. Legend Classification of Materials

0.0  Algae Coated Sealed Surface

Highly Fragmented with Calcite coated
Fracture Surfaces

Maximum Depth of Front Damage

1.0

2.0

3.0

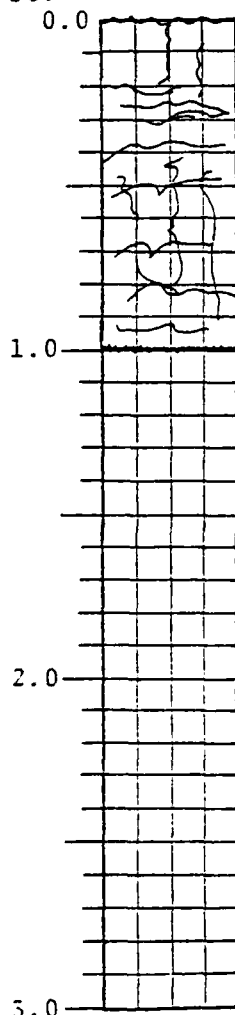
Dresden Island Lock and Dam
 Illinois Waterway, Chicago District
 152mm. (6-in.) Diameter Horizontal Concrete Core

DI WES D-11-77

2.5 ft. Downstream from Upstream Edge of Top of Pier
 South Side Pier 8

Depth Legend
 ft.

Classification of Materials



Not Finished Surface

3-in. Maximum Size Aggregate
 Natural Carbonate Coarse Aggregate W/
 Some Igneous Rock
 Natural Siliceous Fine Aggregate
 Not Air Entrained
 Some Entrapped Air

Maximum Depth
 of Frost Damage

Fractures and breaks are
 through aggregates and paste
 and are parallel to sub-
 parallel.
 All Fractures and breaks are
 partially or entirely coated
 with calcite.

Dresden Island Lock and Dam
Illinois Waterway, Chicago District
152mm. (6-in.) Diameter Horizontal Concrete Core

DI WES D-6-77

8.5 ft. Upstream of Edge of Top of Pier

Depth ft. Legend Classification of Materials

0.0 Finished Surface 1-in. Maximum Size Aggregate
Natural Carbonate Coarse Aggregate w/
Some Igneous Rock Siliceous Fine Aggregate
Some Entrained Air Few Entrapped Air
Light Gray Paste Vertical Fracture

1.0 1/4-in. Steel Rod
1/4-in. Steel Rod Good Tight Contact of Old and New Concrete
Possible Frost Damage Old Lime Coated 5-in. Maximum Size Aggregate Like Above
Tan Paste Fracture of Old Concrete Near Contact
Not Air Entrained

2.0 Found Void with silica gel in new concrete
Index less than 1.49 amorphous

3.0

APPENDIX A: ABBREVIATIONS

Dol - Dolomite
Sh - Shale
Ch - Chert
Cl - Clay
Chy - Cherty
Sty - Stylolitic Bed
Interb - Interbedded
Sf - Soft
Inc - Inclusion
Lyr - Layer
Nod - Nodule
W/ - With
V - Very
Vert - Vertical
Slg - Slightly
Mod - Moderately
Fi - Fine
Bl - Blue
Blk - Black
Br - Brown
Gry - Gray
Grn - Green
Drk - Dark
Fr - Fracture
Ptg - Parting
Jt - Joint
SB - Structural Break
BP - Bedding Plane
Prob MZ - Probably Missing Zone
FA - Fine Aggregate
Nat - Natural
Conc - Concrete
Pc - Piece
Const - Construction
Lt - Light
Gr - Grain

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Stowe, Richard L

Concrete and rock tests, major rehabilitation, Dresden Lock and Dam, Illinois Waterway, Chicago District, Phase I rehabilitation / by R. L. Stowe, G. S. Wong, B. A. Pavlov. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.

47, [9] p., 14 leaves of plates : ill. ; 27 cm.
(Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; SL-80-8)

Prepared for U. S. Army Engineer District, Chicago, Chicago, Illinois.

References: p. 47.

1. Concrete cores. 2. Concrete tests. 3. Core drilling.
4. Dresden Island Lock and Dam. 5. Rock cores. 6. Rock foundations. 7. Rock tests (Laboratory). I. Wong, Jing Sam, joint author. II. Pavlov, Barbara A., joint author.
III. United States. Army. Corps of Engineers. Chicago District.
IV. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; SL-80-8.
TA7.W34m no.SL-80-8

48